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LARGE WEB OPENINGS IN PRESTRESSED CONCRETE T-BEAMS

by

RALPH A. LINDER

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The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies and Research, for acceptance, a thesis entitled LARGE WEB OPENINGS IN PRESTRESSED CONCRETE T-BEAMS submitted by RALPH A. LINDER in partial fulfilment of the requirements for the degree of Master of Science in Engineering.

TO CARROLL

ABSTRACT

This investigation is the third in a continuing program to study the behaviour of and to develop design procedure for prestressed concrete T-beams containing large web openings. The testing was carried out in the I.F. Morrison Structural Engineering Laboratory of the University of Alberta under the supervision of Dr. J. Warwaruk.*

In this test series, thirty prestressed concrete T-beams were tested, all of which contained large web openings. The beams all had a height of 20 inches, a flange width of 20 inches, and a simply supported span of 16 or 20 feet.

The prime variable in the test program was the reinforcing requirements in the region around large web openings. Other parameters such as geometry, loading conditions, and flexural capacity were varied to place different demands on the reinforcement. The reinforcement in the region of an opening was grouped into four types: post reinforcement, solid shear span shear reinforcement, strut shear reinforcement, and strut flexural reinforcement.

The results of the tests are presented in the form of graphs, tables and photographs.

A design procedure is outlined, however, further investigation is recommended.

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CHAPTER 1

INTRODUCTION

1.1 General Remarks

In the construction of a building a reduction in storey height will result in a reduction in overall construction costs. The storey height can be reduced by passing the mechanical ducts transversely through the webs of supporting members rather than hanging them below. This is a relatively simple operation when the supporting members are open web steel joists but for steel and concrete beams extra considerations are required. Much research has been conducted into the design and analysis of steel beams with web holes but at this time only limited research has been carried out on prestressed and reinforced concrete beams with web openings. This research does not indicate a general design procedure for prestressed and reinforced concrete beams; it does, however, indicate that the design and construction of such members is possible, practical and economical.

In a beam with multiple holes in the web, the elements above and below a hole are called struts and the elements between the holes are called posts as shown in Figure 1.1. If the beam is simply supported, the top struts are in compression and the bottom struts are in tension. The forces acting on the struts to either side of the posts are shown in Figure 1.2(a). It can be seen that the hole does not

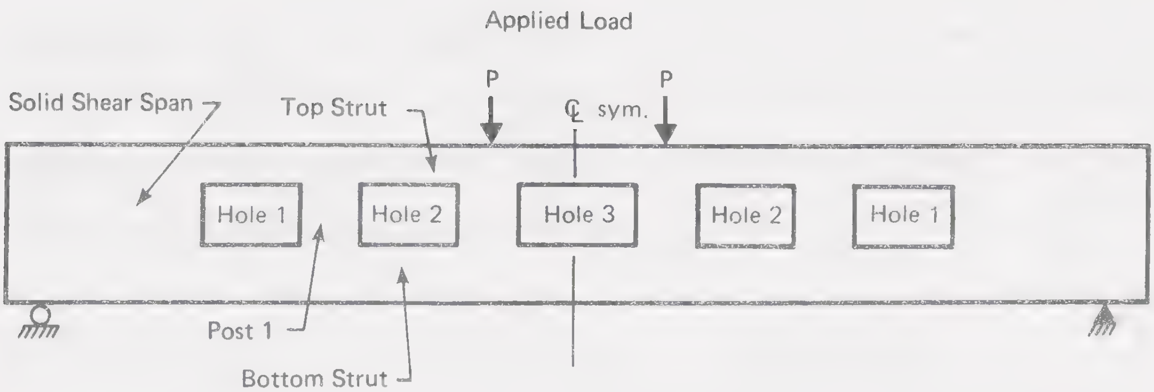
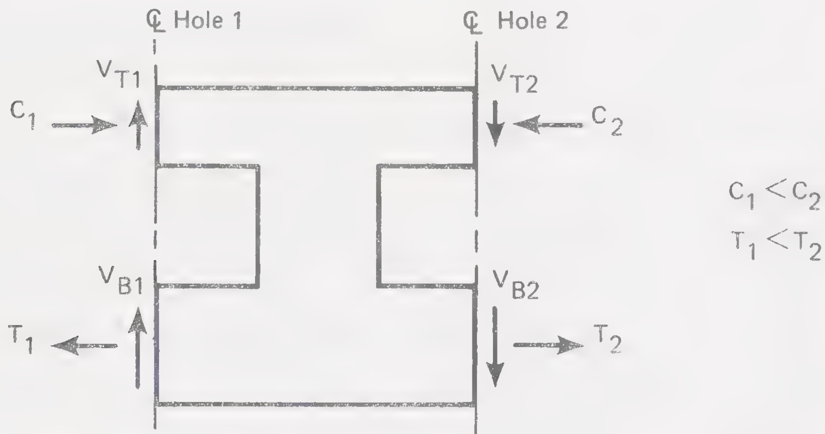
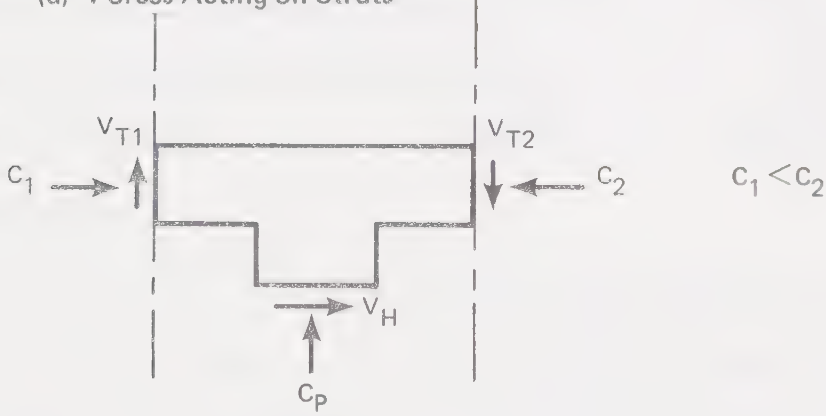


FIGURE 1.1. Holes in Beams



(a) Forces Acting on Struts



(b) Forces Acting on Posts

FIGURE 1.2. Forces Acting on Struts and Posts

drastically affect the tension and compressive forces. However, the holes reduce the section resisting the applied shear. This strut shear produces secondary moments in the struts. The forces acting on a post between two large web openings are clearly seen in the freebody in Figure 1.2(b). The principal force acting on the post is a horizontal shear, V_H, which is equal to the horizontal resultant of the tension or compression forces acting on the struts to either side of the post.

1.2 Scope and Objectives

Some research has been carried out in various countries on the design of prestressed and reinforced concrete beams with large web openings but still much research is required to completely understand and predict the behaviour of such beams. The results of the tests of J. Sauve (12)* and E. LeBlanc (6) on prestressed concrete T-beams containing large, multiple, rectangular and parallelogram shaped openings at the University of Alberta, together with the results of tests by other researchers on tests of similar beams formed the basis for this present series.

This test series continues the work of Sauve and LeBlanc to investigate the behaviour of and to develop design procedures for prestressed concrete T-beams containing large web openings. The main concern of this series was the reinforcing requirements in the region of a large web opening. This reinforcement can be grouped into four types: (i) solid shear span reinforcement, (ii) strut shear reinforcement, (iii) strut flexural reinforcement, and (iv) post reinforcement.

* Number in brackets refers to entries in the reference list.

For each type several arrangements were examined. Other variables were the type of loading, flexural capacity, shape of the holes, the horizontal dimension of the post and holes, and span length.

Thirty simply supported prestressed concrete T-beams with large web openings and symmetrical loadings were tested. The behaviour of each beam was recorded in the form of Demec strain gage readings over the depth of the section, deflection at the beam centerline and below the load points, and strain readings from electrical resistance strain gages mounted on the prestressing strand and on the mild steel reinforcement. Photographs of the beams were also taken.

The main objectives were to study the reinforcing requirement in the region of an opening, to study the effects of various reinforcing arrangements on the behaviour, and to develop a design procedure for prestressed concrete T-beams with large web openings.

CHAPTER 2

REVIEW OF PREVIOUS STUDIES

2.1 Introduction

The American Concrete Institute Standard 381-71 Building Code Requirements for Reinforced Concrete (1) makes no specific recommendations for the analysis and design of beams with large web openings. Some guidance is found in the literature which suggests that the design of reinforced concrete beams with large web openings is possible, practical, and economical. A review of some of these papers is presented here in chronological order.

2.2 Reinforced Concrete T-Beams with a Web Openings

Lorentsen (7) of the Royal Institute of Technology, Stockholm, conducted and published the results of an analytical and experimental study of reinforced concrete T-beams with a single large web opening in 1962. Four beams were tested which correlated well with his analysis. The holes in his test beams were very large, having a depth of 0.53 of the beam height, and a length of 3.0 times the beam height or 0.29 of the clear span. He recommended a design procedure which treats the bottom strut as a tension link and the top strut as the top chord of a Vierendeel truss, although he does not call it that. The total shear was assumed to be carried by the top strut. The solid portion was designed to resist normal beam forces with extra stirrups placed near the hole.

He pointed out that holes in simply supported beams should be placed near the mid span where the normal forces in the struts are large and the shear forces are small, resulting in a minimum principal tensile stress in the concrete. Lorensten also noted the high shear strength of the concrete in the top strut where the axial load is high. He concluded his procedure could be used for beams with multiple openings.

2.3 Prestressed Concrete T-Beams with Large Web Openings

Ragan and Warwaruk (9) in 1967 published the results of tests on prestressed concrete T-beams with large web openings. Their research was carried out at the University of Alberta on four model beams and two full size beams. Three of the model beams and one of the full size beams had large web openings. The design of the full size beam with openings was based on the results of the model tests. Some of their observations and conclusions were:

1. Cracking extended vertically downward from approximately the center of the bottom struts. Therefore the strands in the full size beam with openings were distributed almost evenly across the vertical dimension of the bottom strut.

2. Severe cracking at the connection of the post and the flange led to the provision of reinforcement in the posts with an area 1.3 percent of the horizontal post area.

3. All of the failures were due to inclined cracking in the lower struts, therefore the full size beam had the lower strut reinforced

with U-stirrups spaced at 6 to 12 inches.

4. The mode of failure of two of the model beams was by the formation of a mechanism over two openings. The other beam failed by the formation of a mechanism over one opening.

5. None of the beams with openings failed in a flexural manner; the full size beam failed by shear compression of the top strut.

2.4 Rectangular Concrete Beams with Large Web Openings

In 1967, Nasser, Acavalos and Daniel (8) of the University of Saskatchewan published a report on 10 beams, 9 of which had large rectangular web openings. A simple design procedure was developed, based on four assumptions which were proven adequate for their tests. Their assumptions were:

1. The top and bottom struts behave similar to the chords of a Vierendeel panel.

2. The struts, when they are not subjected to applied loads have contraflexure points at their mid span.

3. The struts, when adequate stirrups are provided, carry shear in proportion to their cross-sectional areas.

4. There is a diagonal force concentration at the corners of the openings equal to twice the simple shear force.

The first assumption was substantiated by the general behaviour of the struts. The second was confirmed by strut deflection measurements and concrete strain measurements at the strut quarter

points. The third was substantiated by concrete strain measurements at the mid height of the struts by electrical resistance strain measuring rosettes. The last assumption was substantiated by plotting the strain in the corner reinforcement versus shear on the section. The strains were normalized to a constant area of reinforcement which was not published. This, together with the fact that the calculations published in the design example, the diagonal force is taken as $2(\sqrt{2} \times V_u)$, makes it uncertain as to whether the stress concentration in the special corner reinforcement is 2 or $2\sqrt{2}$ times the applied shear force.

2.5 Prestressed Concrete T-Beams with Large Web Openings

J. Sauve (12) in 1970 conducted tests on 9 prestressed concrete T-beams with rectangular openings in the Structural Engineering Laboratory of the University of Alberta. The major variables of his program were: spacing of two point loads, vertical shear reinforcement, longitudinal reinforcement and supplementary lower strut shear reinforcement. Some observations and conclusions from his program were:

1. Concentration of the shear reinforcement, required by the ACI (318-71) (1) in a beam without web openings, into the posts of a beam with web openings does not give the beam enough shear capacity to fail in flexure.

2. Any additional shear reinforcement provided in the posts served to increase the load carrying capacity of beams with large openings by 15 to 22 percent.

3. Additional post reinforcement also confined the failure to the struts.

4. A minimum amount of inclined shear reinforcement placed in the lower strut caused the failure to be localized in the posts.

5. The addition of both post and strut shear reinforcement resulted in a redistribution of the stresses in the shear span so that all sections were more equally stressed in diagonal tension.

6. A considerable increase in the supplementary longitudinal reinforcement in the struts did not significantly increase the shear capacity of the beams.

7. A decrease in the number of openings in the shear span increased the shear capacity of the beams.

2.6 Prestressed Concrete T-Beams with Large Web Openings

E. LeBlanc (6) in 1971 conducted tests on ten prestressed concrete T-beams, nine of which contained multiple rectangular or parallelogram shaped openings, in the Structural Engineering Laboratory of the University of Alberta. The main parameters studied in these tests were: the shape of the openings (rectangular and parallelogram), loading conditions (various two point loads and seven point loading), prestress force and shear reinforcement in the posts and lower struts. Five of the beams tested failed in a flexural manner, the remaining five beams failed in shear. Some observations and conclusions from this study were:

1. In prestressed concrete T-beams containing web openings, parallelogram shaped openings accompanied by inclined shear stirrups produced a beam with a higher ultimate shear capacity, relative

to rectangular shaped openings with vertical shear stirrups.

2. The shear design requirements for a two point loading system are more severe than those for a seven point loading system.

3. An increase in the prestress force must be accompanied by an increase in shear reinforcement in order to attain the flexural capacity of a beam without first failing in shear.

4. Since shear failures occur through the struts, shear reinforcement in the posts is not sufficient in itself to prevent shear failure.

5. The possibility of a beam withstanding more severe shear stresses and ultimately failing in flexure is increased by reinforcement in the lower struts below the web openings in the shear span. This reinforcement also produces a more ductile failure in the beam should it fail in shear.

6. Upper web and flange shear reinforcement above the openings in the shear spans would probably increase the possibility of a flexural failure. This is because shear failures tend to initiate in the flange and upper web in this high shear region.

7. The ultimate load and moment calculated using the approximate ACI (1) equation (Equation 18-3) and the manufacturer's guaranteed minimum ultimate strength for the strand is very conservative relative to the actual ultimate load and moment obtained from the tests.

8. The ultimate deflection of beams with openings, failing in flexure, is higher than the same beam with no openings due to the decreased stiffness of a beam with web openings.

In conclusion No. 7 above, it should be noted that in the calculation for strength made by Le Blanc the contribution of the supplementary non-prestressed reinforcement was omitted.

2.7 Square Openings in Webs of Continuous Joists

J.M. Hansen (5) published a report in 1969 which discussed the results of twenty-three tests on full scale one-way joists in negative bending. The principal variables were size location and reinforcement of the openings. Tentative design procedures were discussed. Some of his observations and conclusions were:

1. The minimum web width should be used to calculate the shear capacity of a tapered web subjected to negative bending.

2. Large square openings reduced the strength of the test specimen. An opening of 3/4 of the web depth reduced the strength of the test specimen by two thirds.

3. Moving a square opening in an unreinforced web closer to the support increased the strength of the test specimen.

4. Moving a square opening in an unreinforced web from mid-depth towards the tension fiber did not affect the strength but decreased the cracking load.

5. A two-legged No. 3 stirrup placed vertically at each side of an opening increased the capacity of the test specimen.

6. The compression force in the compression strut at the centerline of the opening was located at the centroid of the strut.

7. Until cracking the distribution of the shear between the struts above and below the opening was in proportion to the cross-sectional area of the struts.

8. After cracking, the compressive strut tended to carry all additional shear.

9. A conservative prediction of the strength of a specimen with unreinforced holes was obtained by calculating the load causing a tensile crack at an opening.

10. A good prediction of the strength of the specimens with reinforced openings was obtained by calculating the load causing either eccentric shear compression or diagonal tension failure, using the ultimate strength provisions of the 1963 ACI Building Code.

2.8 Reinforcing Requirements for Concrete Beams With Large Web Openings

M. Ramey and D. Tattershall (10) conducted and published in 1973 a theoretical and experimental program from which they developed a working stress design procedure. The theoretical analysis of sixty simply supported beams and thirty-five bent caps was carried out using E.L. Wilson's finite element program. The experimental program consisted of tests on twelve simply supported beams. From their results they concluded:

1. The solid section can be designed in the usual manner.

2. The design of the struts assuming Vierendeel behaviour is adequate.

3. Beams containing multiple openings can be designed as single openings if the posts have a horizontal dimension of at least half the height of the beam.

4. There is a diagonal stress concentration at the corners of the openings. The value of this stress is obtained from curves developed from the finite element analysis and is always less than the total shear on the section for moment-shear ratios of less than 4.

J.M. Hanson presented a discussion of the paper which questioned the validity of the elastic analysis used and pointed out the restrictions of the rectangular section tested.

2.9 Circular Openings in Webs of Continuous Beams

Somes and Corley (13) published a report in 1974 on tests of 19 full scale joists in negative bending. The tests were conducted at the Portland Cement Association Structural Development Laboratory. Twelve of the joists had singular openings, three had multiple openings, and four had no openings. Based on their observations, they presented design recommendations for continuous one-way joists with circular openings in the negative moment region. The highlights of their design recommendations are:

1. The minimum web width should be used to calculate the shear capacity in the negative moment region.

2. Small openings (openings less than 0.25 the web depth in diameter) require no reinforcement and can be placed anywhere in the web,

provided they are no closer to the extreme compression fiber than the depth of the equivalent compression block.

3. No opening should be placed so that it encroaches on the equivalent rectangular stress block.

4. Large openings (openings greater than 0.25 of the web depth in diameter) should be reinforced with vertical stirrups on each side of the opening. The area of the stirrups required on each side of an opening is calculated by dividing the shear capacity of the section by the yield strength of the stirrups.

5. The minimum post width is 0.25 of the web depth or 4 inches, whichever is the greater.

6. Large web openings, particularly multiple openings, reduce the stiffness of a floor joist.

CHAPTER 3

EXPERIMENTAL PROGRAM

3.1 Introduction

This test series is one in a continuing program at the University of Alberta to investigate the behaviour and develop design procedures for prestressed concrete T-beams with large web openings. The prime concern is the study of the overall behaviour of such members and particularly the effect of reinforcement in the region around the web openings. A general discussion of the experimental program is presented in the following sections. A complete description is found in the Appendix.

3.2 Beam Geometry

In this series, 30 prestressed concrete T-beams were tested; 27 of these had rectangular openings and 3 had parallelogram shaped openings. The simply supported span length was either 16 or 20 ft., but the different span length is not considered as a prime variable as it was changed to facilitate casting. The overall cross-section of the beams was constant for all beams tested. The flange was 20 inches wide and 2 inches thick. The stem of the "T" was 4 inches wide and the overall height was 20 inches. The web openings were all 8 inches high with their mid-height located 1.17 in. below the neutral axis of the gross concrete cross section. Figure 3.1 shows the concrete cross-section and the vertical location of the holes. The rectangular open-

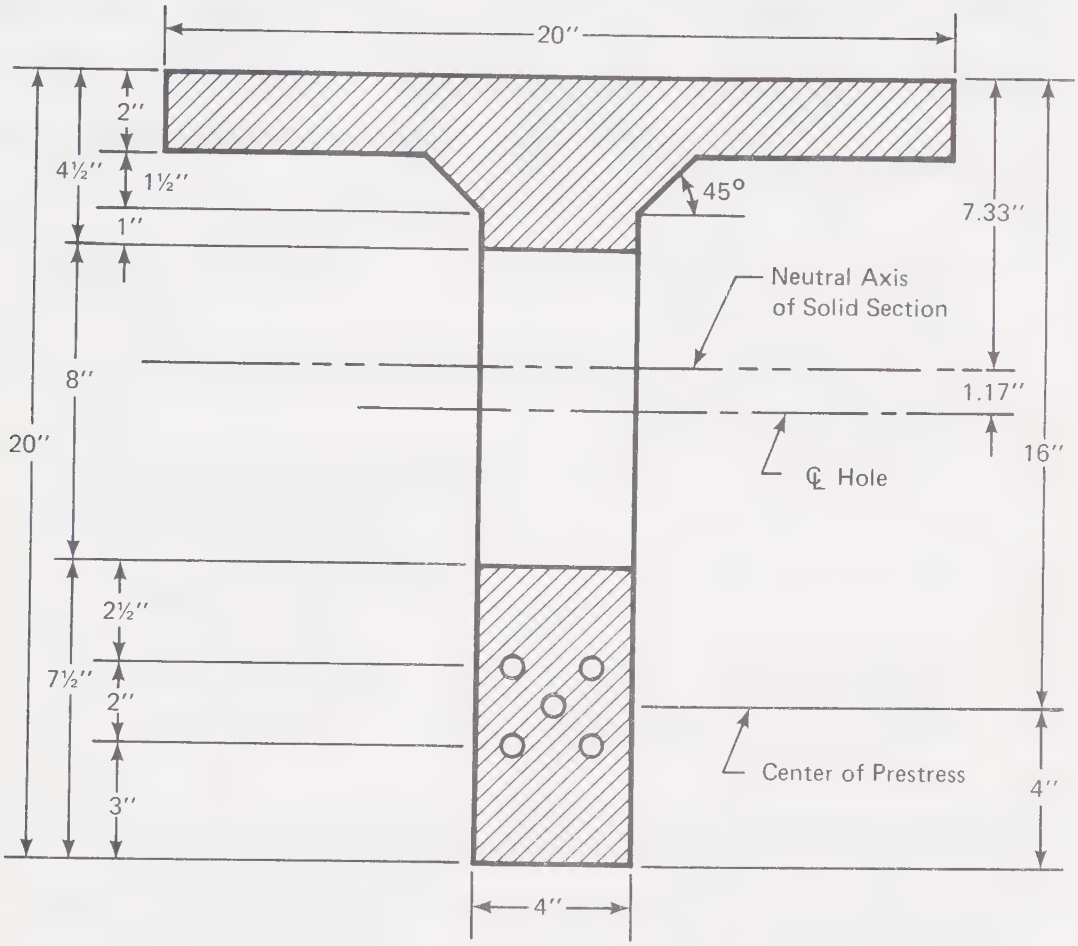


FIGURE 3.1. Typical Concrete Cross Section at an Opening

ings had lengths from 12 inches to 26 inches separated by posts with a horizontal dimension of 8 inches to 12 inches. The transverse area between the post centerlines was reduced 20 to 29 percent. The parallelogram shaped openings were 16 inches long and separated by posts with a horizontal dimension of 8 inches. The transverse area between the posts centerlines was reduced 25 percent.

3.3 Reinforcement

The reinforcement was varied in five main areas:

1. The pure moment region.
2. The solid shear spans.
3. The top struts.
4. The bottom struts.
5. The posts.

The principal tension reinforcement in the pure moment region was 4 or 5 3/8 inches seven-wire prestressing strands which were continuous throughout the beam. The solid shear spans were reinforced for shear with double legged stirrups of No. 2 or No. 3 bars with various spacings set either vertically or inclined at 45°. The top struts were reinforced to resist strut moments and shear due to Vierendeel behaviour. The top longitudinal reinforcement consisted of four No. 3 bars in all but two beams where four No. 2 bars were used. The bottom longitudinal reinforcement was provided by two No. 2, 3, 4 or 5 bars with 22 beams having No. 3 bars. The top struts in the shear spans of 28 of the beams of this series also had shear reinforcement consisting of closed

No. 2 stirrups set vertically or inclined at 45° with spacings from 1.2 in. to 4.00 in. The bottom struts had supplementary longitudinal reinforcement in the shear spans only. Fourteen beams had two No. 3 bars in the bottom only, fourteen beams had two No. 2, 3, 4, or 5 bars in the top and bottom, and two beams were cast without supplementary longitudinal reinforcement in the bottom struts. These two beams also had the shear capacity of the bottom strut reduced by casting into the struts four oiled vertical metal plates and eliminating the bond on the prestressing strands by wrapping them with polyethylene throughout the strut length. This was done to evaluate the proportion of the total shear over the cross-section carried by the top and bottom struts. The posts were reinforced with vertical or inclined double legged stirrups with the same shape as those used in the solid shear spans. In the shear spans multiple stirrups were used and in the pure moment region the posts had one stirrup. In five beams, supplementary post reinforcement was added to the main vertical stirrups of post 1. The supplementary reinforcement in four beams consisted of closed horizontal stirrups, and in one beam, two inclined No. 5 bars. These reinforcement details are shown in Table 3.1 and Figures 3.2, 3.3 and 3.4.

3.4 Construction

The fabrication of the test specimens was carried out in the I.F. Morrison Structural Engineering Laboratory of the University of Alberta. First, the reinforcement cages were tack welded and set in place on the prestressing bed. Next the prestressing strands were threaded through the cages and prestressed. The two halves of the forms

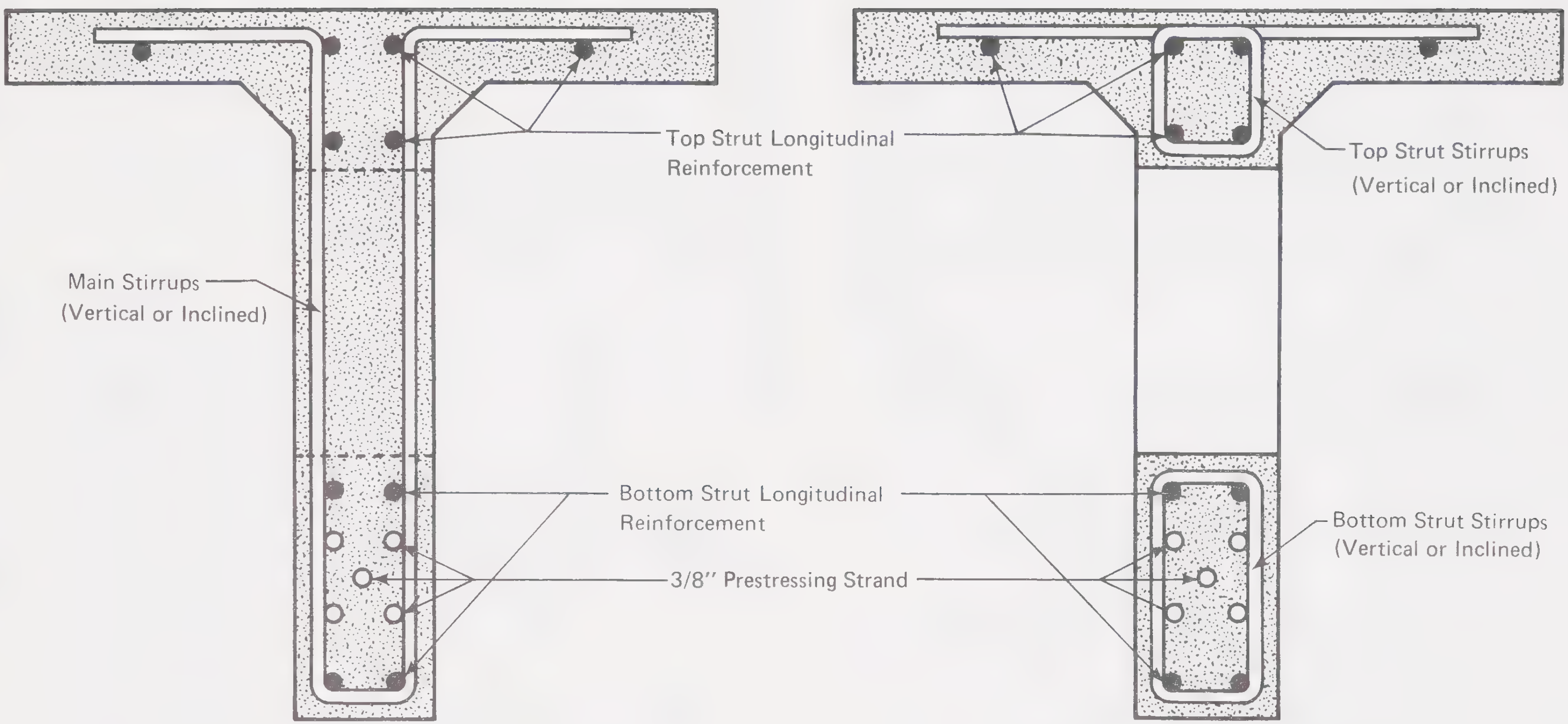
TABLE 3.1 REINFORCEMENT DETAILS

Beam No.	Top Strut Reinforcement			Bottom Strut Reinforcement			Post Reinforcement		Solid Shear Span Avg. Spacing (in)	Number of 7-Wire Strands	Opening Length (in)	Horizontal Post Length (in)	Opening Shape	Shear Span (in)
	Flexure		Shear	Flexure		Shear	Main Stirrups	Supplemental Post Reinforcement						
	Top	Bottom	Spacing (in)	Top	Bottom	Spacing (in)								
1-16-6*	4#3	2#3	3.5**	-	2#3	3.5**	3#3**	-	8	4	16	8	Rectangular	72"
2-16-4	4#3	2#3	3.5	-	2#3	3.5	3#3	-	8	4	16	8	Rectangular	48
3-16-6	4#3	2#3	3.5	-	2#3	3.5	3#3	-	8	5	16	8	Rectangular	72
4-16-6	4#3	2#3	3.5	-	-	-	3#3	-	8	4	16	8	Rectangular	72
5-16-4	4#3	2#3	3.5	-	-	-	3#3	-	8	4	16	8	Rectangular	48
6-16-6	4#3	2#3	3.5	-	2#3	3.5	5#3	-	6	4	16	8	Rectangular	72
7-16-6	4#3	2#3	3.5	-	2#3	3.5	5#3	-	6	5	16	8	Rectangular	72
8-16-7L	4#3	2#3	3.5	-	2#3	3.5	3#3	-	8	4	16	8	Rectangular	7 @ 2'
9-16-7L	4#3	2#3	3.5	-	2#3	3.5	3#3	-	8	5	16	8	Rectangular	7 @ 2'
10-16-6	4#3	2#3	3.5	-	2#3	3.5	3#3	2#5 @ 45°	8	4	16	8	Rectangular	72
11-16-4-P	4#3	2#3	3.5 @ 45°	-	2#3	3.5 @ 45°	3#3 @ 45°	-	8 @ 45°	5	16	8	Inclined	48
12-16-6-P	4#3	2#3	3.5 @ 45°	-	2#3	3.5 @ 45°	3#3 @ 45°	-	8 @ 45°	5	16	8	Inclined	72
13-16-6-P	4#3	2#3	3.5 @ 45°	-	2#3	3.5 @ 45°	2#3 @ 45°	-	12 @ 45°	5	16	8	Inclined	72
14-12-6	4#3	2#3	-	-	2#3	-	3#3 @ 45°	-	8 @ 45°	5	12	12	Rectangular	72
15-12-6	4#3	2#3	-	-	2#3	-	3#3	-	8	5	12	12	Rectangular	72
16-12-6	4#3	2#3	4.0	-	2#3	4.0	3#3	-	8	5	12	12	Rectangular	72
17-16-4	4#3	2#3	1.875	2#3	2#3	1.5	4#3	-	15	5	16	8	Rectangular	48
18-16-4	4#3	2#5	1.875	2#5	2#5	1.5	4#3	-	15	5	16	8	Rectangular	48
19-16-6	4#3	2#3	1.875	2#3	2#3	1.5	4#3	7#2	15	5	16	8	Rectangular	72
20-26-5	4#3	2#3	2.5	2#3	2#3	1.875	4#3	-	15	5	26	12	Rectangular	60
21-26-5	4#3	2#5	2.5	2#5	2#5	1.875	4#3	-	15	5	26	12	Rectangular	60
22-26-5	4#3	2#3	1.1875	2#3	2#3	2.75	4#3	-	15	5	26	12	Rectangular	60
23-16-4	4#3	2#5	1.875	2#5	2#5	1.5	4#3	-	#2 @ 6	5	16	8	Rectangular	48
24-16-6	4#2	2#2	3.0	2#2	2#2	1.625	4#3	-	#2 @ 15	5	16	8	Rectangular	72
25-16-6	4#2	2#2	3.0	2#2	2#2	1.625	4#3	4#3	#2 @ 15	5	16	8	Rectangular	72
26-21-7	4#3	2#3	2.875	2#3	2#3	1.875	4#3	4#3	#2 @ 15	5	21	9 5/16	Rectangular	84
27-16-4	4#3	2#5	1.25	2#5	2#5	1.0625	4#3	-	#2 @ 6	5	16	8	Rectangular	48
28-16-4	4#3	2#5	2.5	2#5	2#5	2.125	4#3	-	#2 @ 6	5	16	8	Rectangular	48
29-12-6	4#2	2#2	3.0	2#2	2#2	1.625	1#2 2#3	-	15	5	12	12	Rectangular	72
30-21-7	4#3	2#4	2.875	2#4	2#4	1.875	3#3	3#3	#2 @ 15	5	21	8	Rectangular	84

* Beam Number Code (x, y, z, p)

- x - Order of testing and casting of the specimens
- y - Hole length in inches
- z - Shear span length in feet or 7L for 7 point loading
- p - Indicates beams with parallelogram shaped openings

** Set vertically or inclined at the given angle



(a) Section Through a Post

(b) Section Through a Hole

FIGURE 3.2. Typical Reinforcement Details

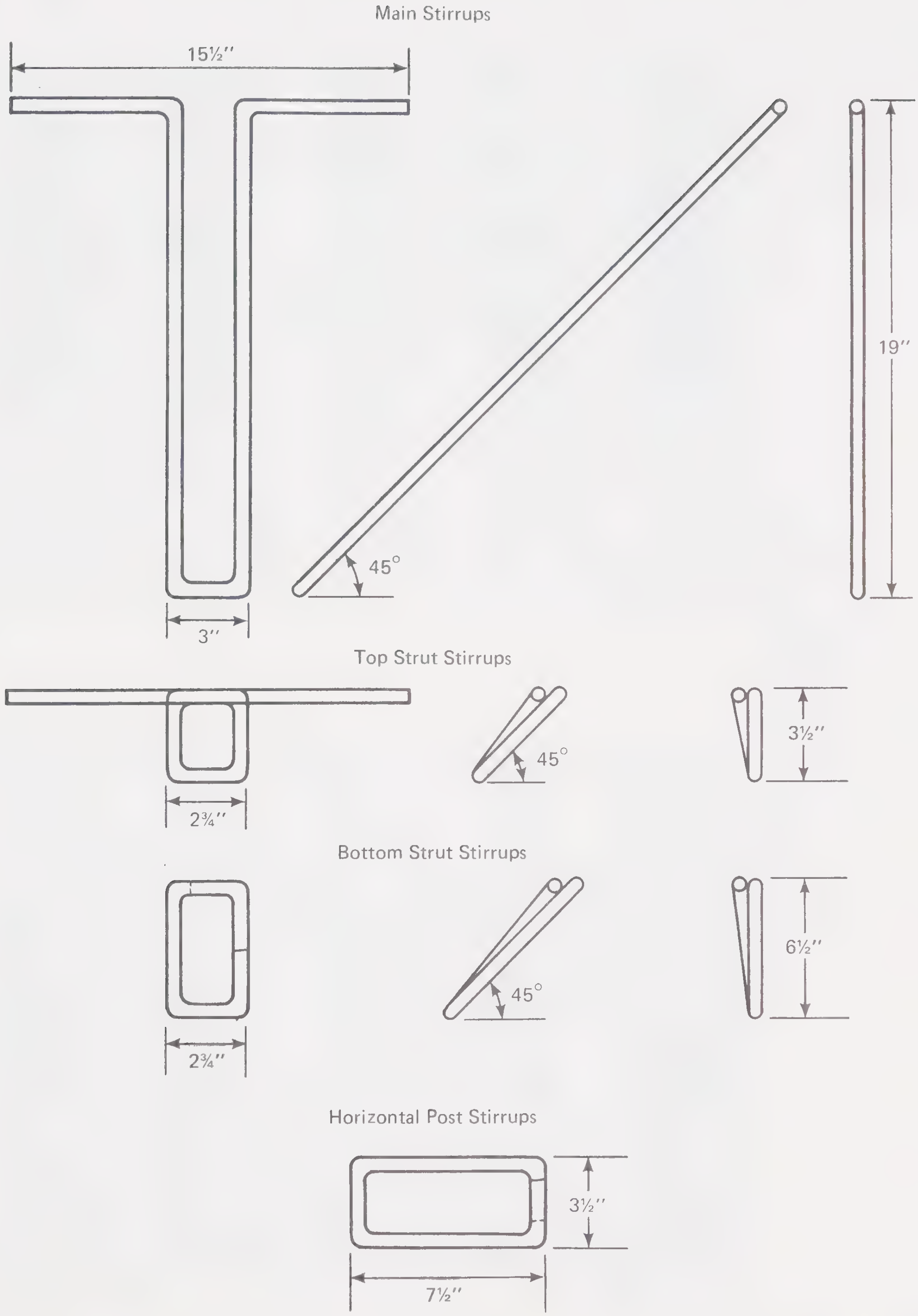


FIGURE 3.3. Stirrup Details

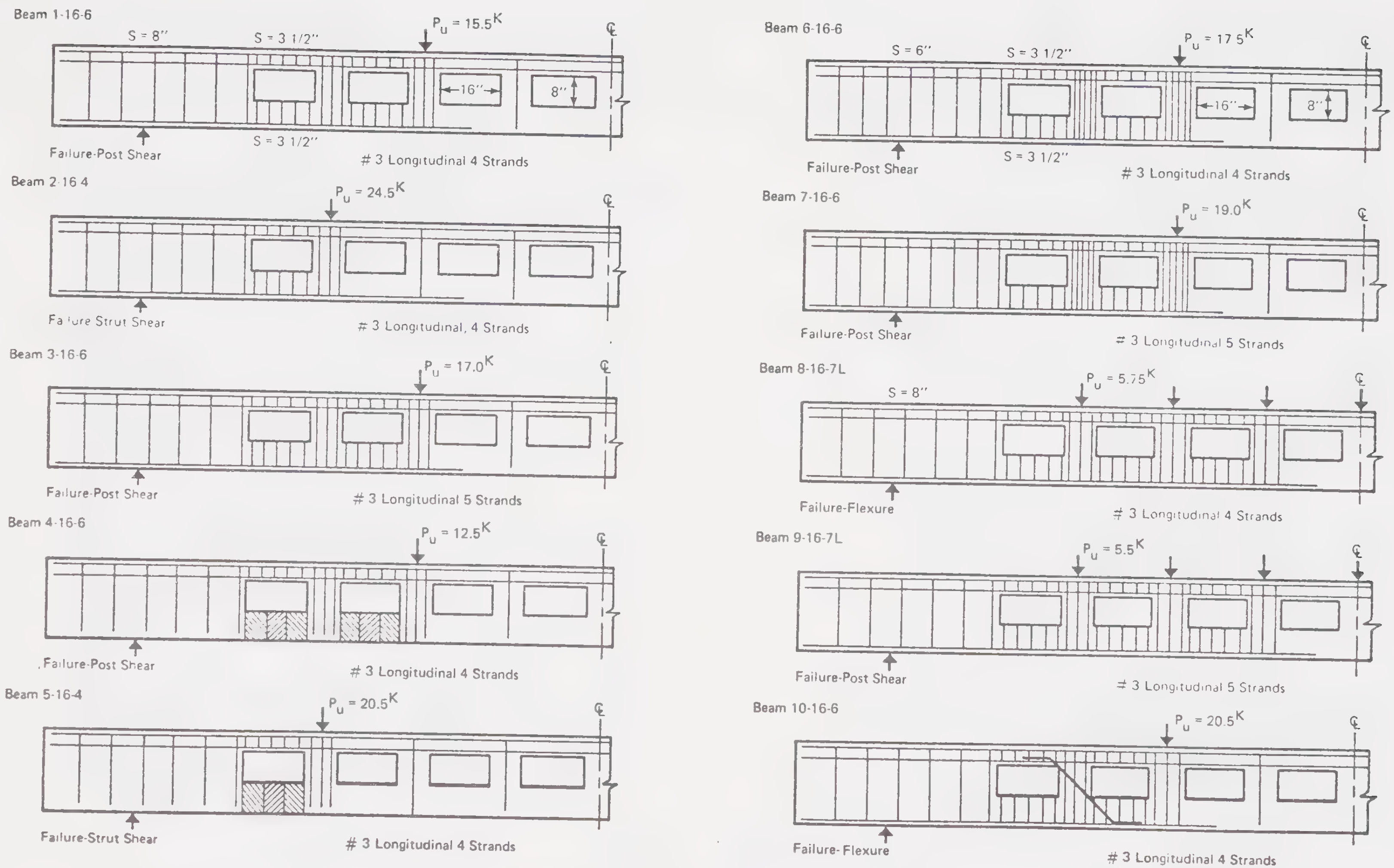


FIGURE 3.4(a). Test Beam Reinforcement Details

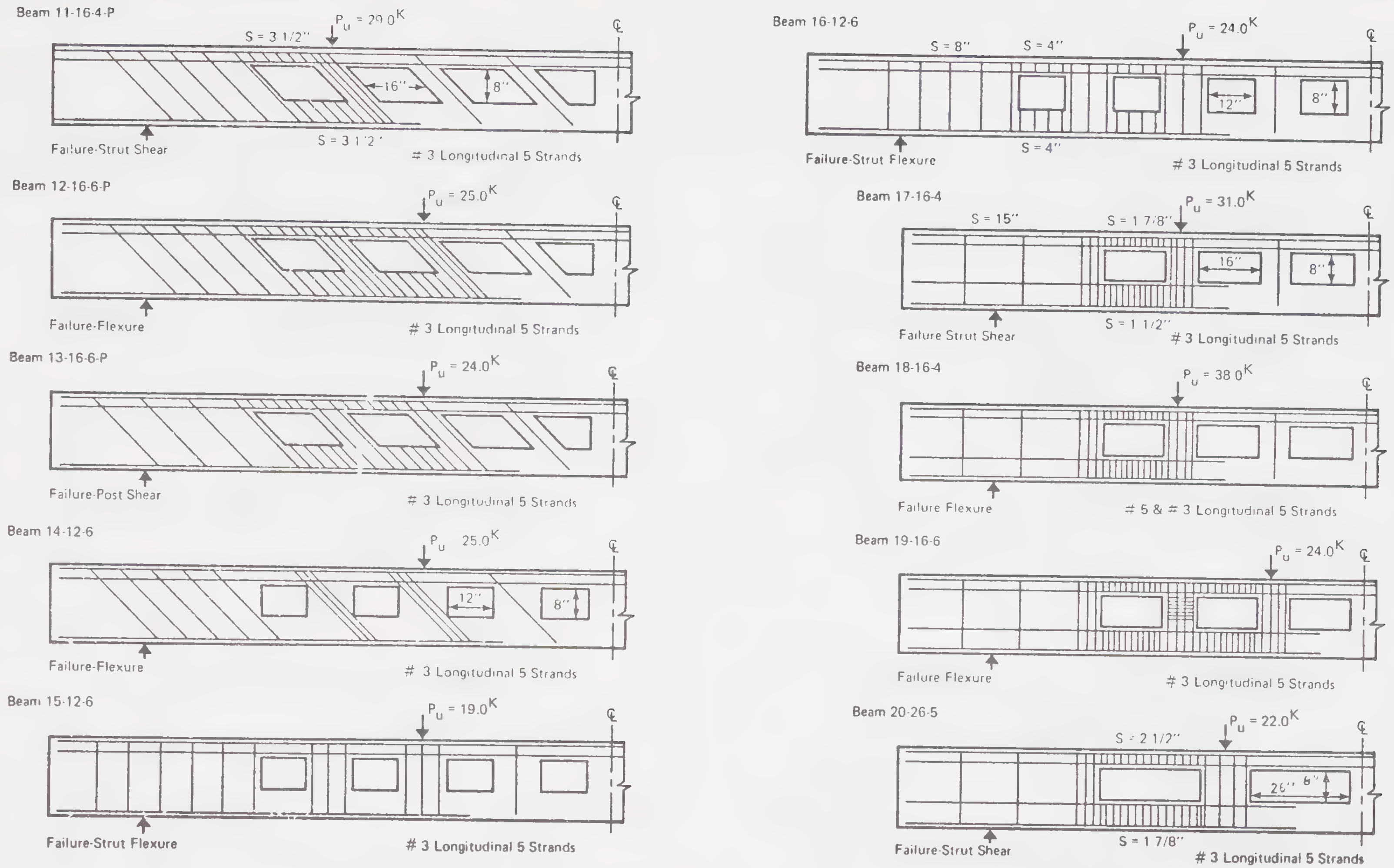


FIGURE 3.4(b). Test Beam Reinforcement Details

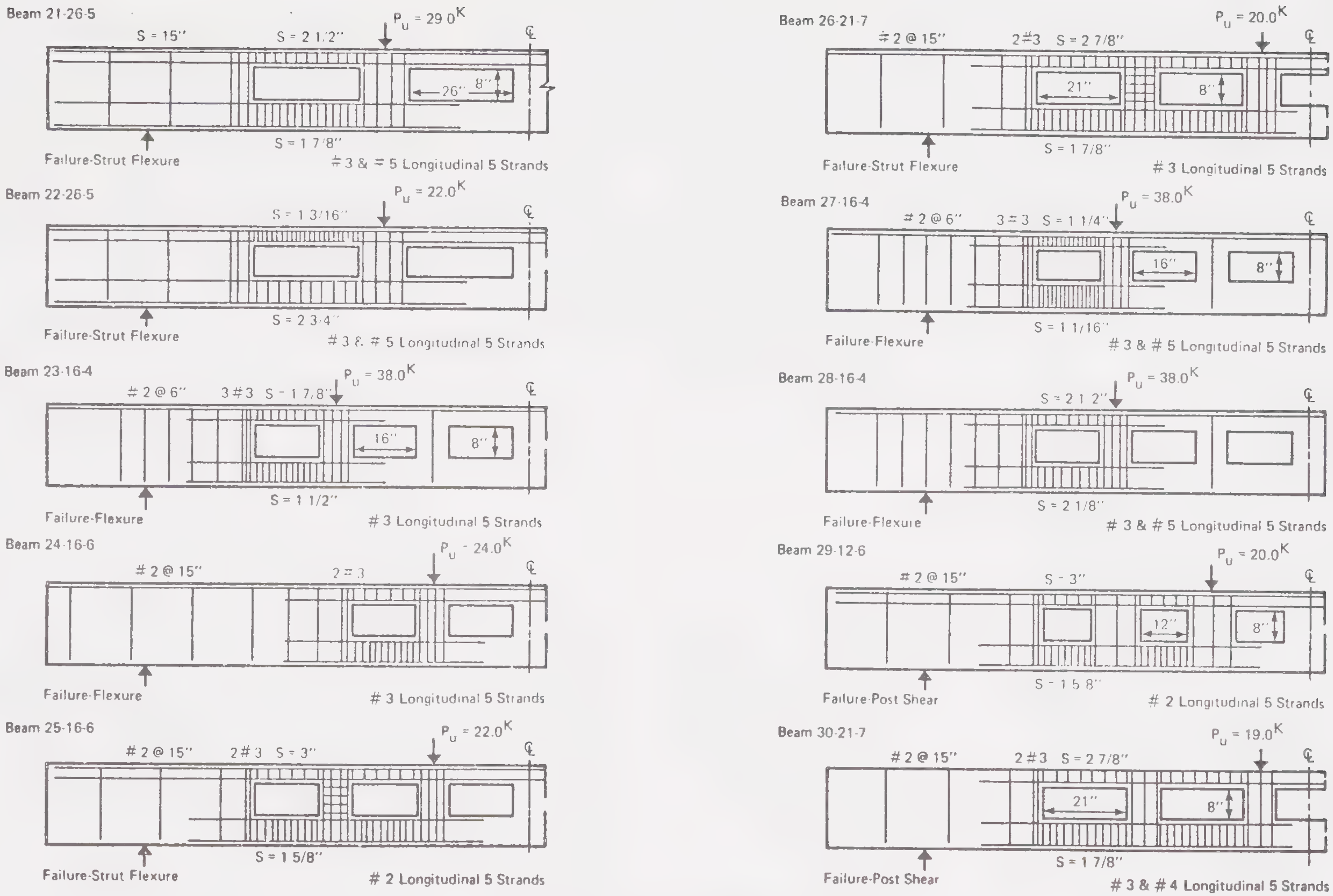


FIGURE 3.4(c). Test Beam Reinforcement Details

were then bolted together with styrofoam blocks forming the voids. The concreting was completed using high-early strength concrete mixed in the laboratory batch plant. The prestressing strands were cut after six days of moist curing. Six by twelve cylinders were cast from each batch of concrete to determine the compressive strength and the tensile splitting strength. A more complete description of the construction details and the materials used is presented in Appendix A.

3.5 Instrumentation

All beams were instrumented in the same manner, using:

1. Electrical resistance strain gages mounted on the mild steel shear and flexural reinforcement and prestressing strands.

2. Demec strain gages fixed to the concrete at the beam centerline.

3. Deflection gages at the beam centerline and below the load points.

The strain gages were mounted and waterproofed on the mild steel reinforcement before the cages were assembled. On prestressing strands, however, the gages were mounted and waterproofed after the initial prestressing was completed. The gages on the prestressing strands were mounted on one of the six curved wires of the seven-wire strand and oriented along the axis of that wire, approximately 8°30' from the longitudinal axis of the strand. The Demec gages at the beam centerline were used to make measurements of deformation which facilitated the calculation of prestressing losses and the centerline strain distribution as the test proceeded. The deflection gages consisted of

a metal ruler with divisions of 0.01 inches hung from the lower portion of the beam. The readings were taken with a precise level located in front of the test specimen. All of the gages except the Demec gages were zeroed just prior to the start of testing. The loads on the beam at this time included the dead weight of the beam and the loading harnesses and the effective prestress force. Typical instrumentation details are shown in Figure 3.5, and the instrumentation for each beam is given in Appendix B.

3.6 Test Setup and Procedure

The first step in the test procedure was to set the beam on beam seats which were fabricated for earlier tests (12). These beam seats rested on the knife edges of a simple support roller system which in turn rested on concrete pedestals. Figure 3.6 shows the beam seat details and Figure 3.7 shows the typical test setup.

Both two point and seven point loadings were applied using loading harnesses. The upper part of each harness rested on the beam through a 5 inch by 5 1/2 inch by 1 inch bearing plate and the lower portion hung below the floor. Hydraulic jacks were placed between the lower part of the harness and the floor. Hydraulic pressure to the jacks was provided by an Amsler pendulum dynamometer and distributed evenly to the jacks through a manifold. In each case the load was applied directly above a post on the longitudinal centerline of the beam. Figure 3.4 shows the loading positions for each beam.

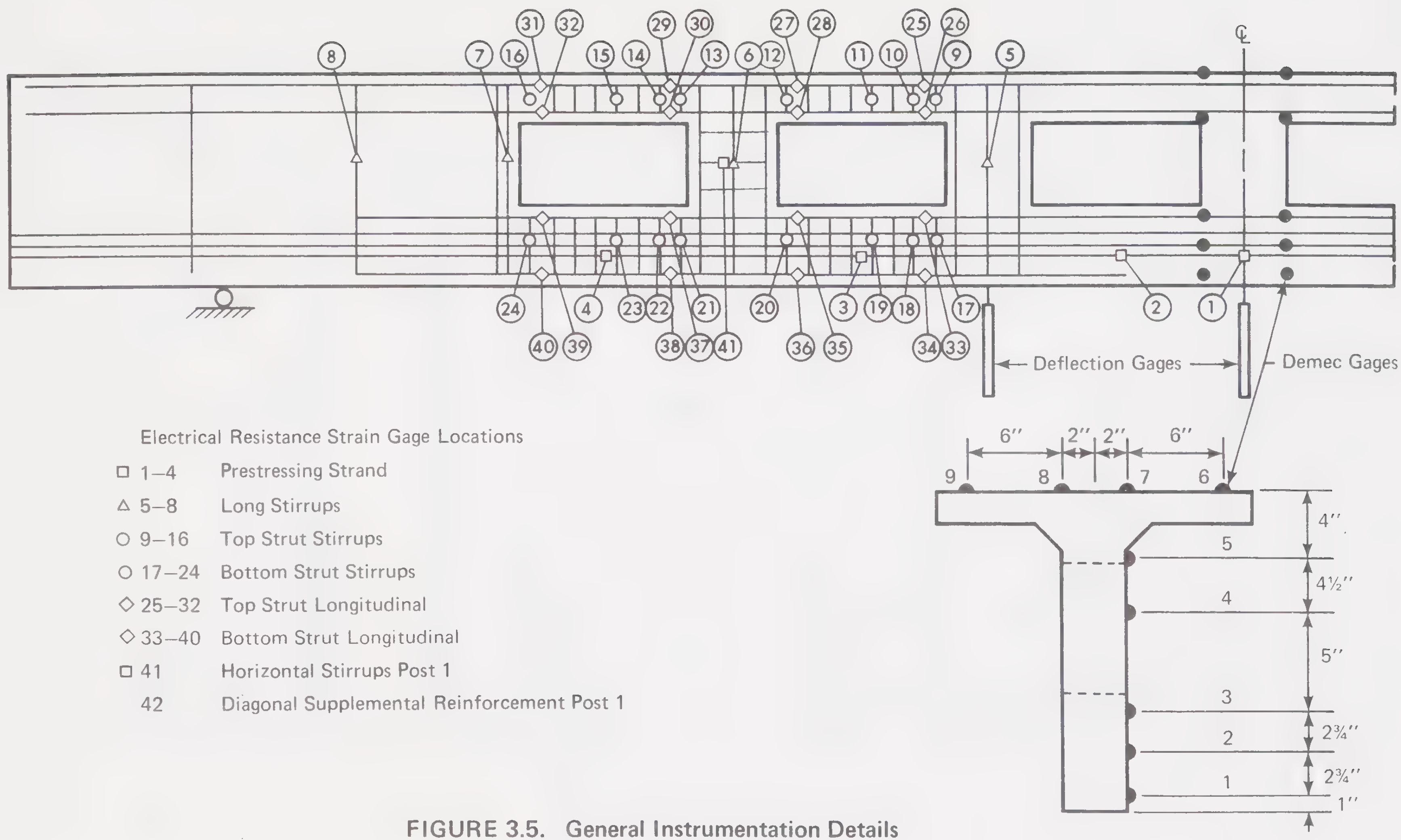


FIGURE 3.5. General Instrumentation Details

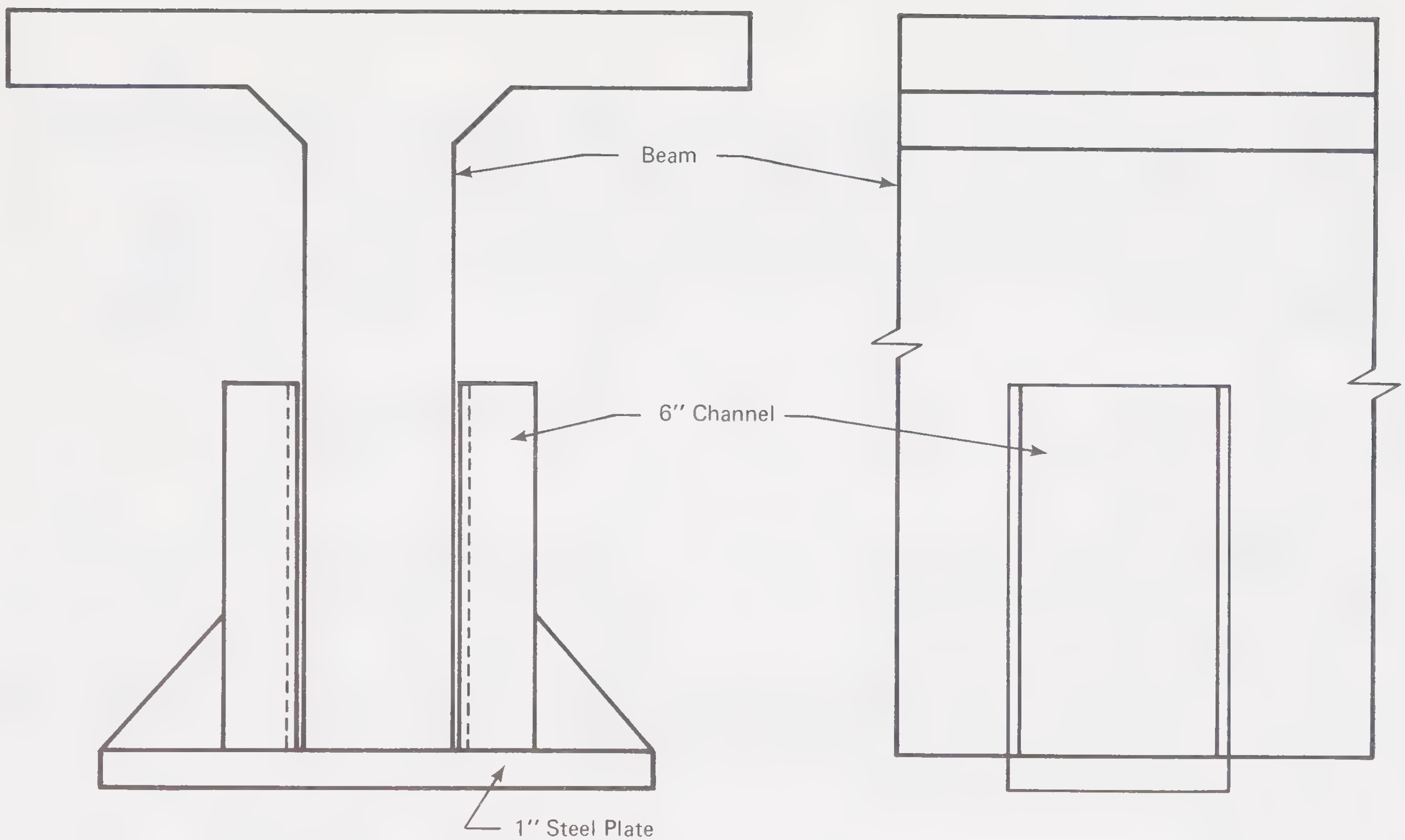


FIGURE 3.6. Beam Seat Details

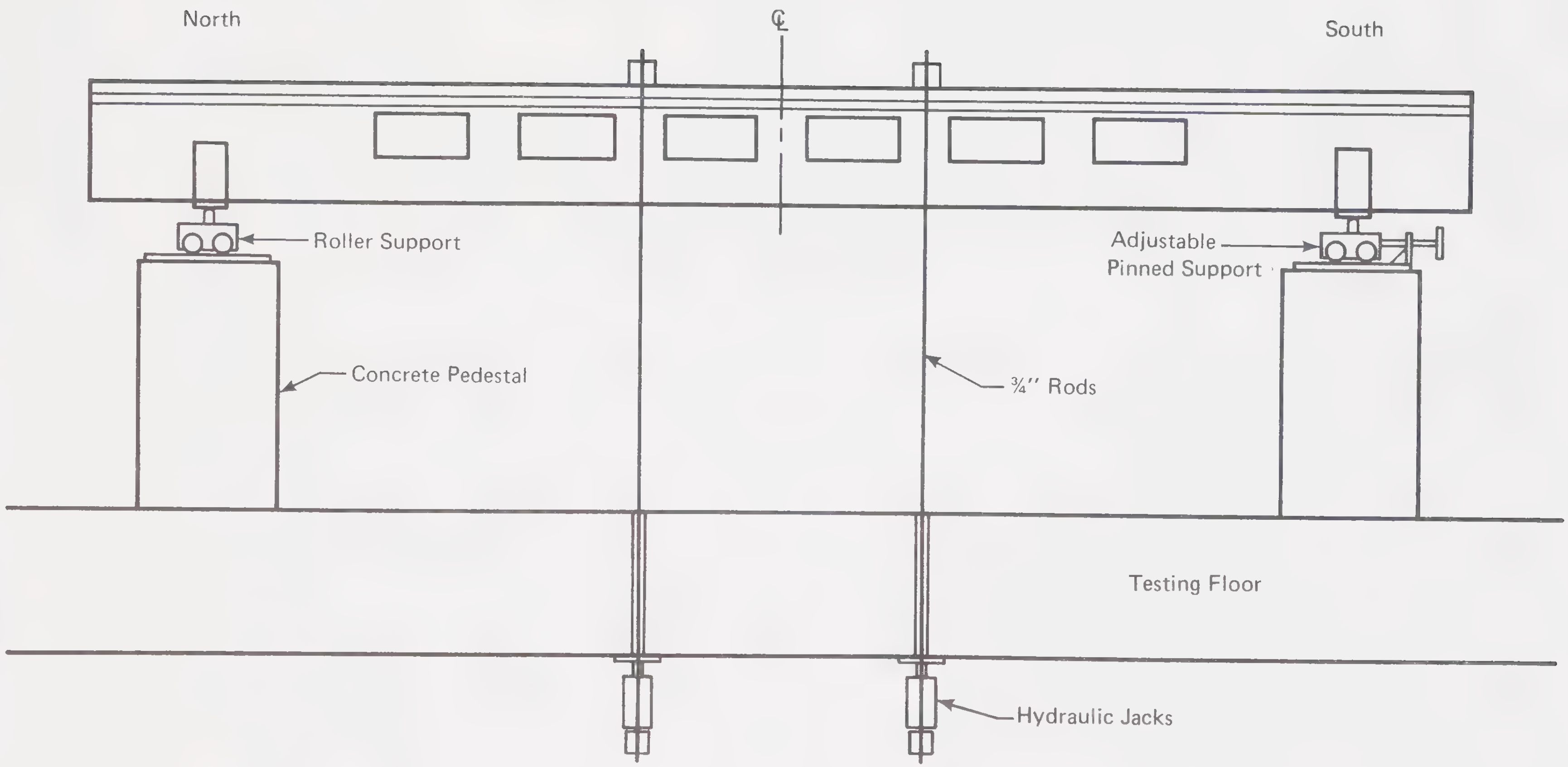


FIGURE 3.7. Typical Test Setup

Each beam was loaded using a sufficient number of increments to facilitate accurate observation and recording of the behaviour. At each load increment readings of the electrical resistance strain gages, deflection gages and Demec gages were taken. All visible cracks were traced on the beam surface and numbered with the load increment number when the crack appeared. To provide a good record of the cracking behaviour, photographs were taken before and after failure.

CHAPTER 4

TEST RESULTS

4.1 Introduction

The tabulated, graphical and photographic results of all the tests are presented in this chapter. They were obtained directly, or calculated from the measurements taken during the testing of the thirty prestressed concrete T-beams. The measurements taken were those of the electrical resistance strain gages, mechanical Demec strain gages and deflection gages. The principal test results are summarized in tabular manner. The graphical results include:

1. Moment-deflection relationships.
2. Moment-strain relationships for the prestressing strand.
3. Load-strain relationships for the shear reinforcement.
4. Load-strain relationships for the strut flexural reinforcement.
5. Moment-strain relationships for the concrete at the beam centerline.

Each plotted curve is the actual recorded behaviour of the gage within the limits of the plot. Points off the plot are indicated by an arrow beside the last plotted value. A complete record of the gage readings is given in Appendix B. The photograph results show the cracking and failure patterns of the beams tested.

4.2 Principal Test Results

Table 4.1 presents a summary of the principal test results

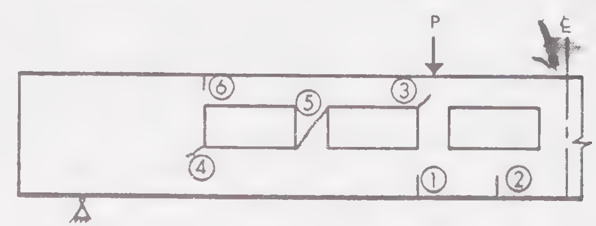
TABLE 4.1 SUMMARY OF TEST RESULTS

Beam Name	Failure Load/ Jack (kips)	Failure Moment (in-k)	Theo. Failure Moment* (in-k)	Test/Theo	Failure Mode	Cracking Loads** 1 (k)	2 (k)	3 (k)	4 (k)	5 (k)	6 (k)	Effective Prestress (k)	ϵ_{Pe} ($\frac{IN}{IN} \times 10^6$)	f'c (psi)	f_{sp} (psi)
1-16-6	15.5	1116	1370	.81	Post Shear	8.0	9.0	7.5	6.0	9.0	-	47.260	5110	5821	476
2-16-4	24.5	1176	1370	.86	Strut Shear	11.0	14.5	9.0	6.0	21.0	19.5	47.014	5084	5906	424
3-16-6	17.0	1223	1680	.73	Post Shear	9.0	10.5	6.0	5.0	8.0	1.6	50.776	4392	5296	424
4-16-6	12.5	900	1370	.66	Strut Shear	-	10.0	8.0	-	7.0	1.1	44.931	4858	5088	371
5-16-4	20.5	984	1370	.72	Strut Shear	-	14.0	8.0	-	17.0	15.5	47.768	5165	5409	392
6-16-6	17.5	1260	1370	.92	Post Shear	8.0	8.0	7.0	6.0	7.0	10.5	44.907	4855	5529	301
7-16-6	19.0	1368	1680	.81	Post Shear	9.0	11.0	6.0	4.0	6.0	1.6	51.623	4466	6348	368
8-16-7L	5.75	1587	1370	1.16	Flexure	2.5	2.5	2.25	2.0	3.0	4.25	45.629	4934	5255	367
9-16-7L	5.50	1518	1680	.90	Post Shear	2.5	2.5	2.0	1.5	2.25	3.75	53.913	4664	5709	365
10-16-6	20.5	1476	1370	1.08	Flexure	8.0	10.0	7.0	6.0	7.0	14.5	46.698	5050	5741	392
11-16-4-P	29.0	1392	1680	.83	Strut Shear	16.0	18.0	2.0	10.0	-	-	53.898	4662	6192	447
12-16-6-P	25.0	1800	1680	1.07	Flexure	10.0	12.0	10.0	11.0	9	-	53.849	4658	5833	404
13-16-6-P	24.0	1728	1680	1.03	Post Shear	8.0	10.0	9.0	9.0	6.0	-	52.199	4515	5394	401
14-12-6	25.0	1800	1680	1.07	Flexure	12.0	11.0	12.0	8.0	15.0	-	53.769	4651	5706	410
15-12-6	19.0	1368	1680	.81	Strut Shear	9.0	9.0	9.0	6.0	13.0	-	56.366	4876	5853	435
16-12-6	24.0	1728	1680	1.03	Strut Shear	9.0	11.0	10.0	8.0	11.0	21.0	54.515	4716	5883	436
17-16-4	31.0	1488	1680	.89	Strut Flex	12.0	17.0	10.0	6.0	17.0	29.0	54.329	4700	5542	466
18-16-4	38.0	1824	1680	1.09	Flexure	10.0	12.5	10.0	7.5	20.0	22.0	53.253	4607	5933	385
19-16-6	24.0	1728	1680	1.03	Flexure	10.0	11.0	10.0	8.0	10.0	22.0	51.868	4487	5875	367
20-26-5	22.0	1320	1680	.79	Strut Flex	7.0	10.0	6.0	5.0	19.0	11.0	50.942	4407	5690	528
21-26-5	29.0	1740	1680	1.04	Strut Flex	6.0	10.0	6.0	6.0	22.5	13.5	50.667	4383	5517	438
22-26-5	22.0	1320	1680	.79	Strut Flex	8.0	11.0	8.0	4.0	14.0	12.0	50.574	4375	5385	360
23-16-4	38.0	1824	1680	1.09	Flexure	12.5	17.0	7.5	7.5	17.0	24.0	53.695	4645	4991	422
24-16-6	24.0	1728	1680	1.03	Flexure	10.0	12.0	8.0	6.0	-	-	56.645	4900	5226	497
25-16-6	22.0	1584	1680	.94	Strut Flex	10.0	12.0	8.0	10.0	8.0	20.0	56.552	4892	5517	376
26-21-7	20.0	1680	1680	1.00	Strut Flex	8.0	9.0	6.0	6.0	6.0	14.0	54.817	4742	5274	340
27-16-4	39.0	1872	1680	1.11	Flexure	10.0	12.0	8.0	6.0	20.0	24.0	47.844	4139	5378	548
28-16-4	38.0	1824	1680	1.09	Flexure	10.0	14.0	6.0	6.0	16.0	20.0	50.619	4379	5469	482
29-12-6	20.0	1440	1680	.86	Post Shear	9.0	10.0	8.0	9.0	12.0	-	53.840	4657	5080	332
30-21-7	19.0	1596	1680	.95	Post Shear	6.0	8.0	6.0	6.0	6.0	17.0	51.715	4474	4766	340

* ACI SECT 10.7

** Crack Locations:
1 Flexure crack in bottom of bottom strut near load
2 Flexure crack in pure moment region
3 Corner crack bottom of top strut near load
4 Corner crack top of bottom strut near load
5 Diagonal crack Post #1
6 Flexure crack top of top strut near reaction

(or see illustration)



including failure loads and moments, theoretical failure moments, failure modes, cracking loads, the splitting and compressive strengths of the concrete, the effective prestress force, and the strain in the strand at the beginning of the test.

The failure loads are the maximum loads per jack carried by the specimen. The failure moments are the moments at the beam centerline calculated from the failure loads and the loading conditions.

The theoretical failure moments are the flexural capacities calculated according to Section 18.7 of ACI 318-71 (1) using $f_{pu} = 275$ ksi (the actual ultimate tensile stress in the strand at fracture), the tensile force developed by two #3 longitudinal bars having a yield strength of 50 ksi and an effective depth of 4 inches, the average concrete strength and $\phi = 1.0$. A sample calculation of flexural capacity is shown in Appendix C.

The cracking loads listed are the loads at which cracks became visible to the unaided eye or began to elongate from cracks due to concrete shrinkage and the effective prestress force.

The effective prestress force and the associated strain in the prestressing strand are included to facilitate the calculation of the force in the prestressing strand at various load increments.

The geometry and reinforcement details are not included in the table but are given in Chapter 3.

4.3 Moment Deflection Relationships

The moment deflection curves are plotted in Figures 4.1 to 4.9. The deflections are those read with a survey level from the centerline deflection gages. The moments have been calculated at the beam centerline from the load indicated by the Amsler loading apparatus and include the weight of the loading harnesses but not the weight of the beam itself.

Curves are plotted for all thirty beams tested in this program and for comparison the moment deflection curves for the control beams of Jaques Sauve (11) and Eric Le Blanc (6) are included. These beams are numbered JS-1 and EL-1 respectively and are plotted in the first plot of beams with similar loading conditions.

The figures are grouped according to loading conditions; Figure 4.1 beams with 7-point loading including EL-1, Figure 4.2 and Figure 4.3 beams with 4 ft. shear spans, Figure 4.4 beams with 5 ft. shear spans, Figure 4.5 to 4.8 beams with 6 ft. shear spans (Figure 4.5 includes JS-1), Figure 4.9 beams with 7 ft. shear spans. The tabulated data for these plots is presented in Appendix B.

4.4 Moment Strain Relationships for the Prestressing Strand

Figures 4.10 to 4.28 are plots of strain in the prestressing strand versus moment at the beam centerline. The strains are those read directly from the electrical resistance strain gages mounted on the prestressing strand at gage locations 1 to 4 as shown in Figure 3.5.

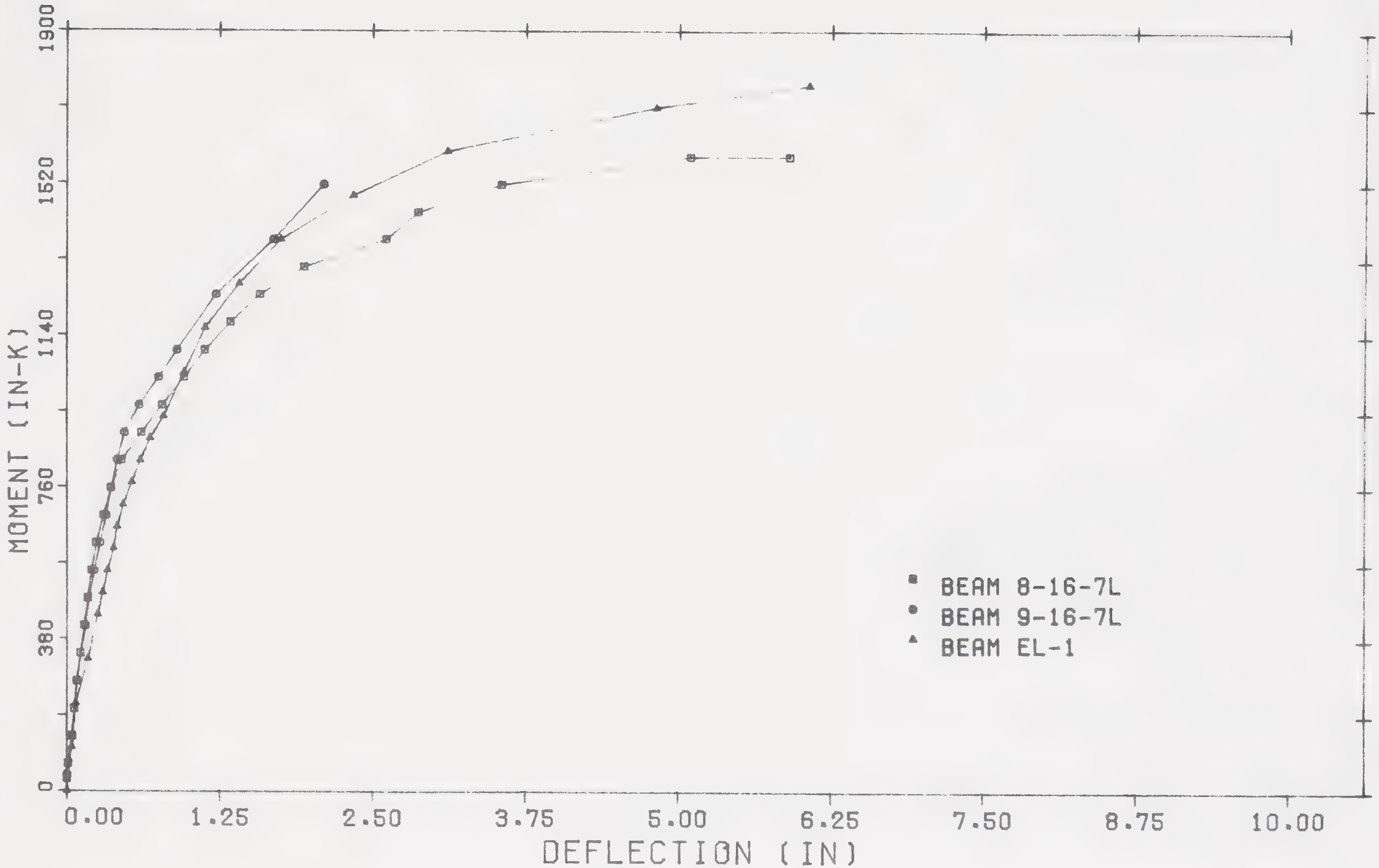


FIG. 4.1 MOMENT DEFLECTION FOR 7-POINT LOADING

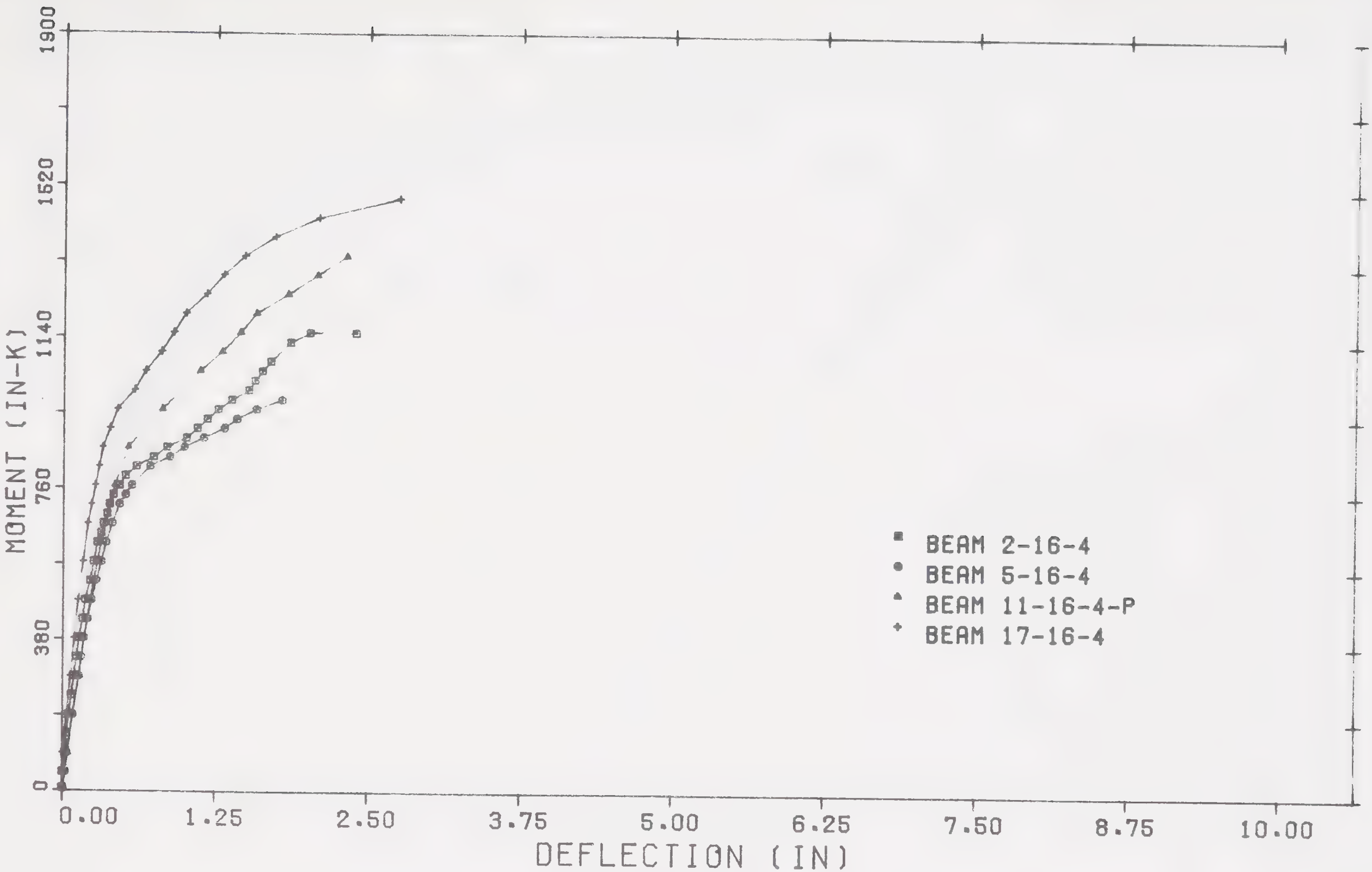


FIG. 4.2 MOMENT DEFLECTION FOR 4FT. SHEAR SPAN

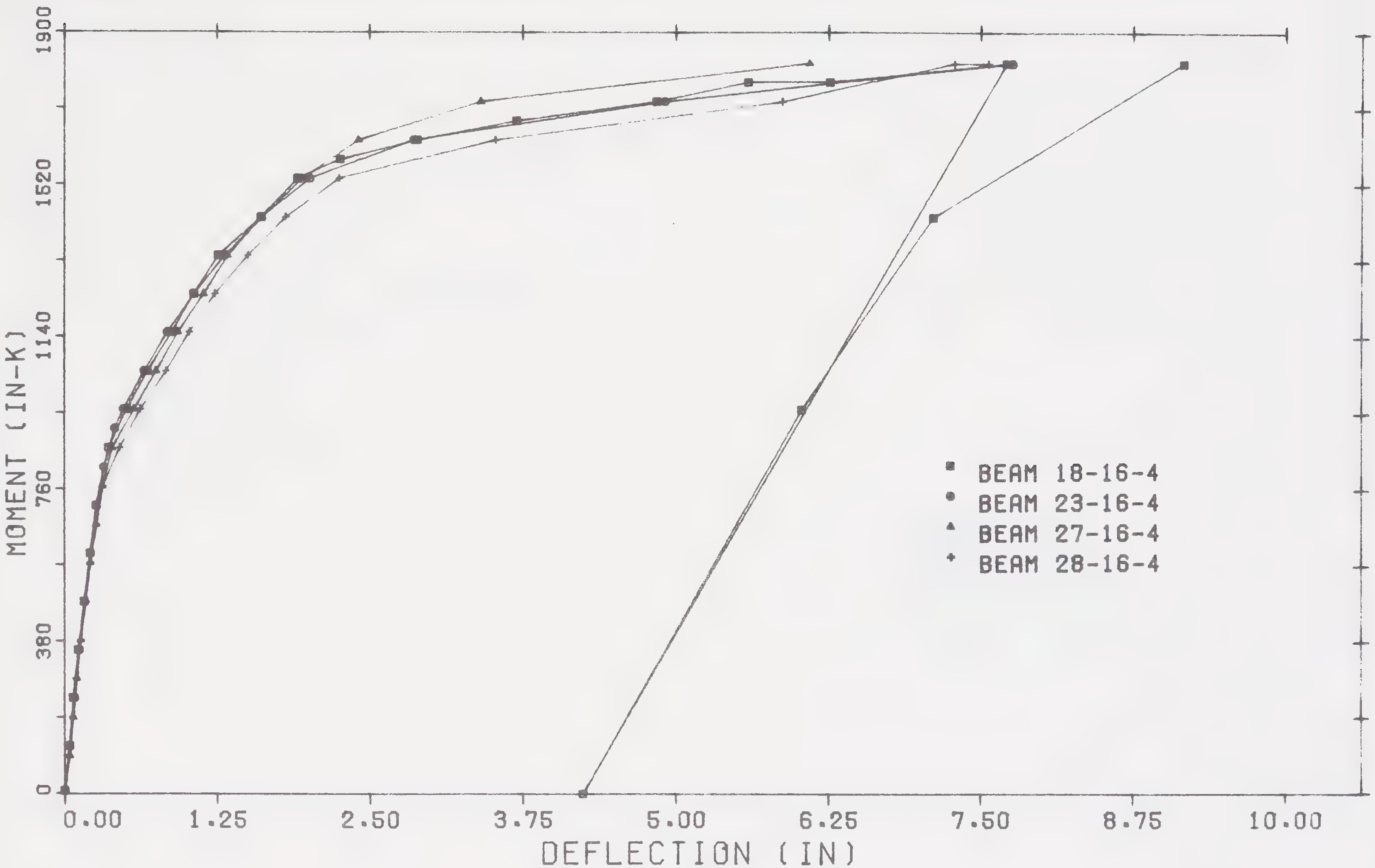


FIG. 4.3 MOMENT DEFLECTION FOR 4FT. SHEAR SPAN

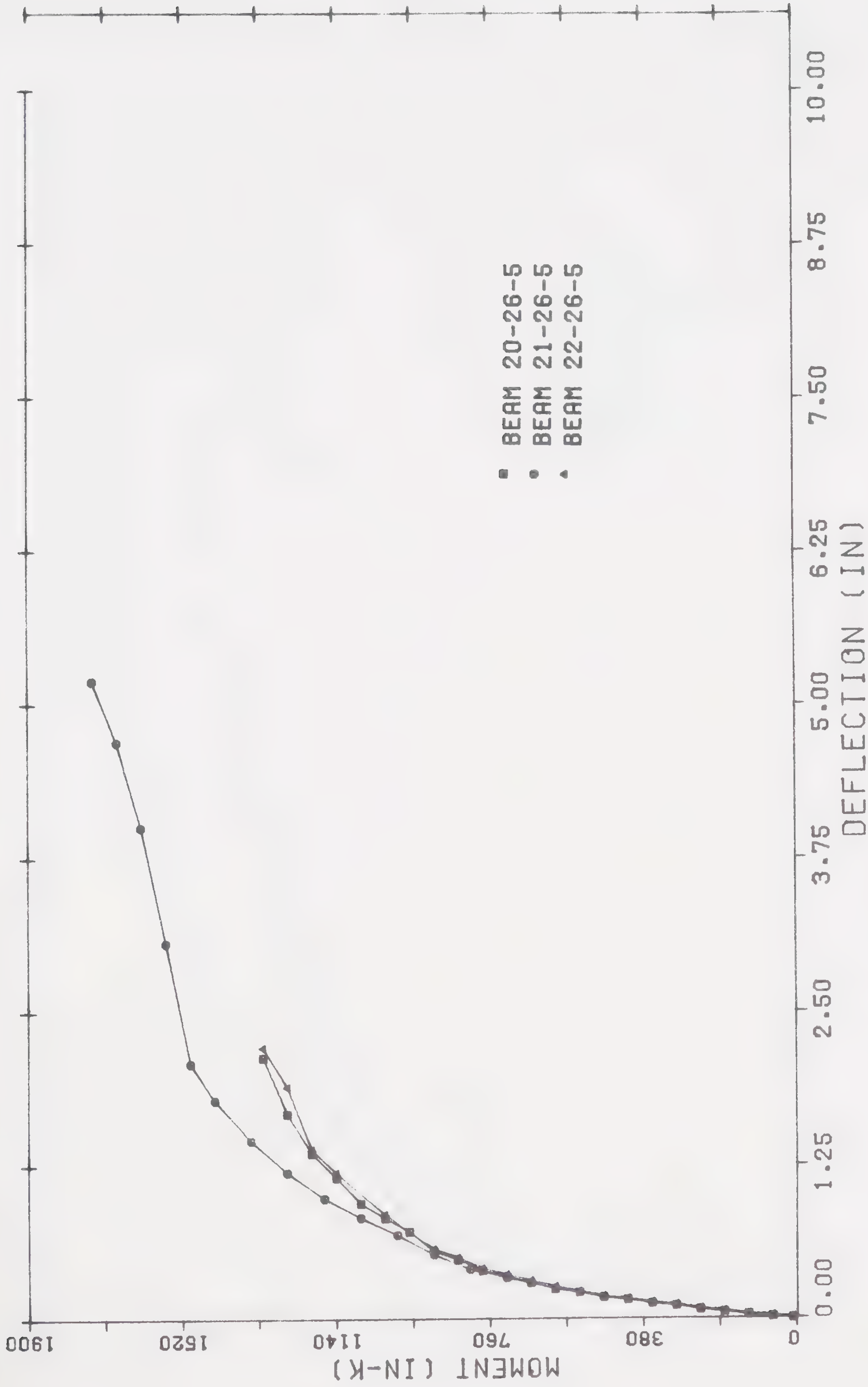


FIG. 4.4 MOMENT DEFLECTION FOR 5FT. SHEAR SPAN

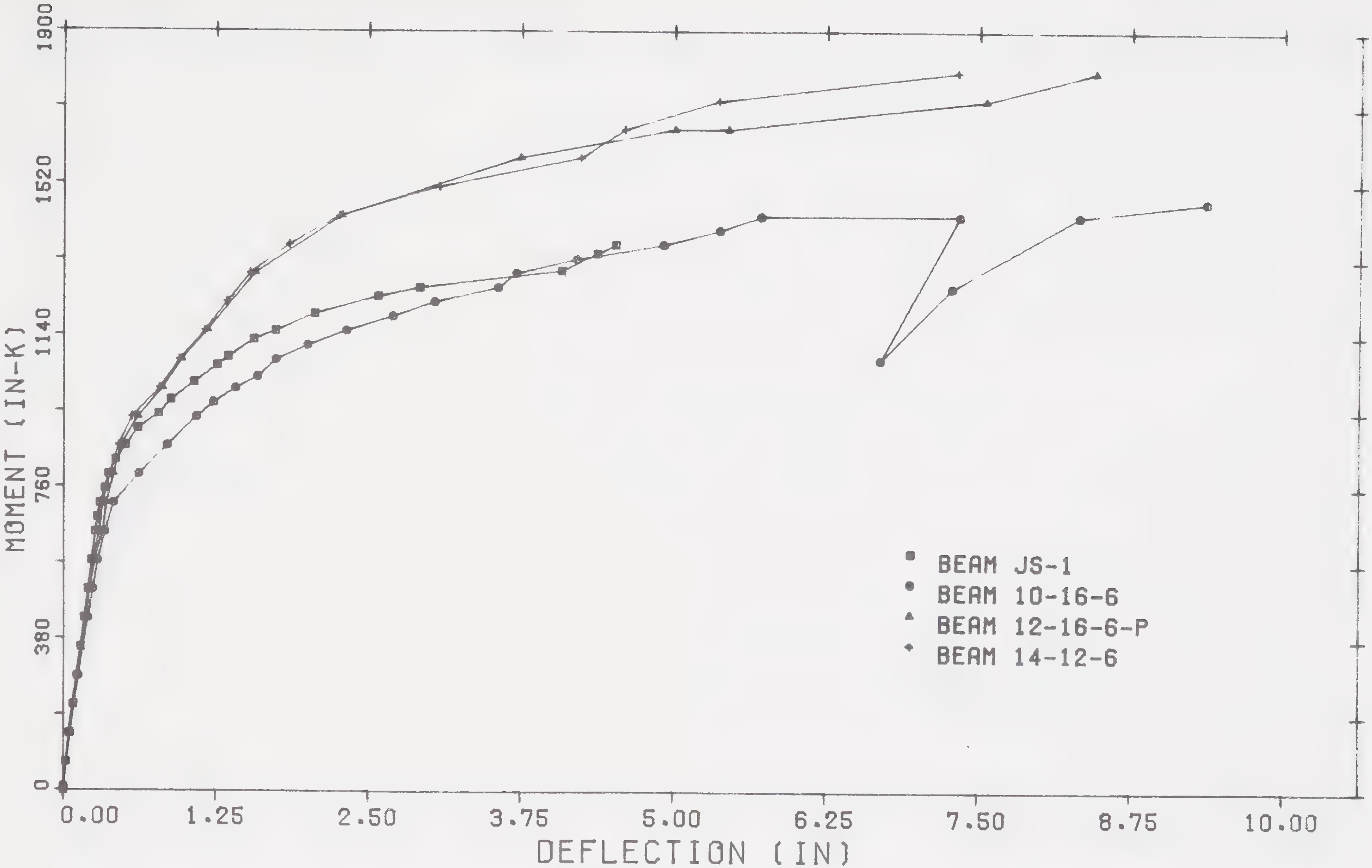


FIG. 4.5 MOMENT DEFLECTION FOR 6FT. SHEAR SPAN

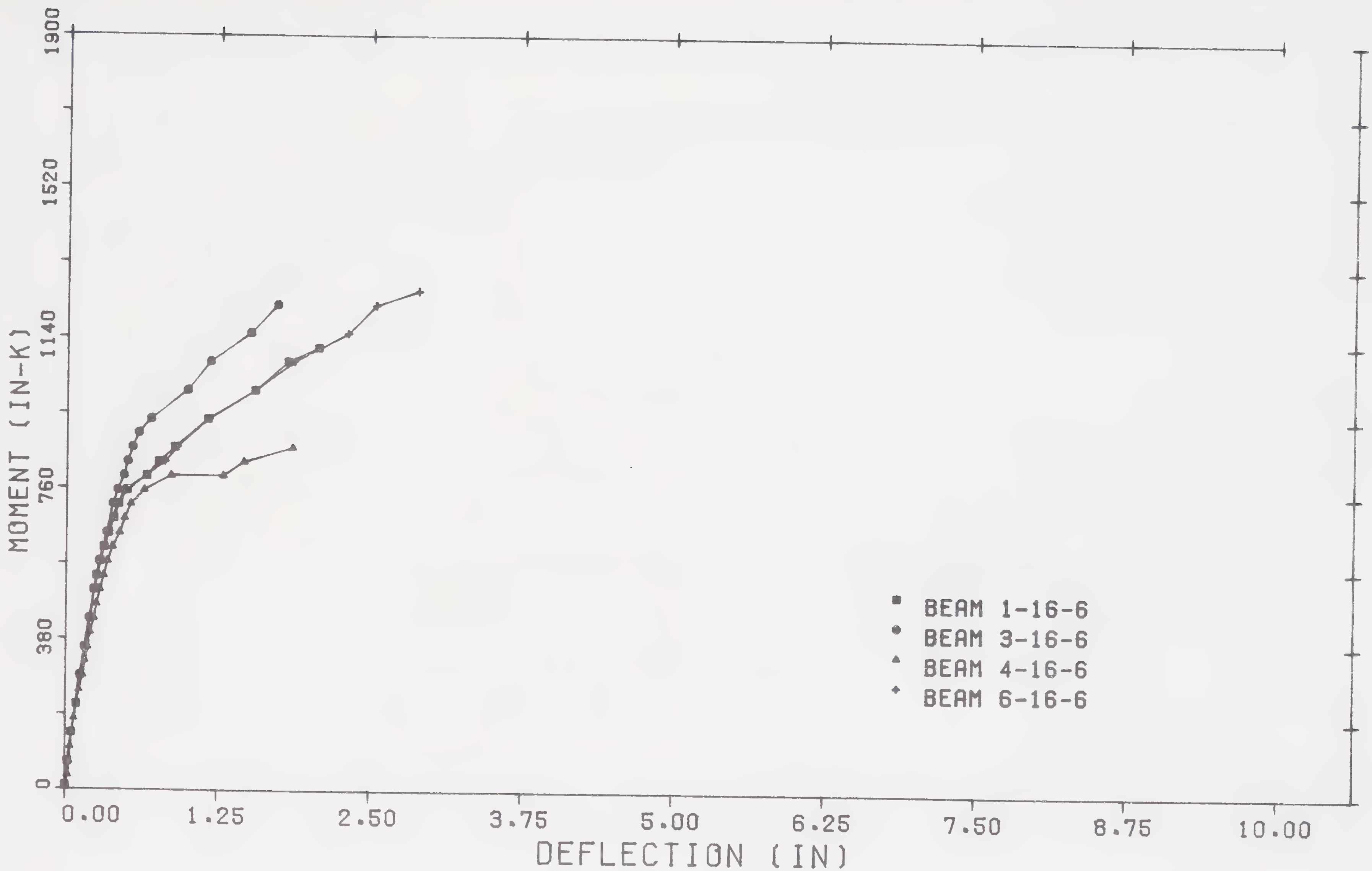


FIG. 4.6 MOMENT DEFLECTION FOR 6FT. SHEAR SPAN

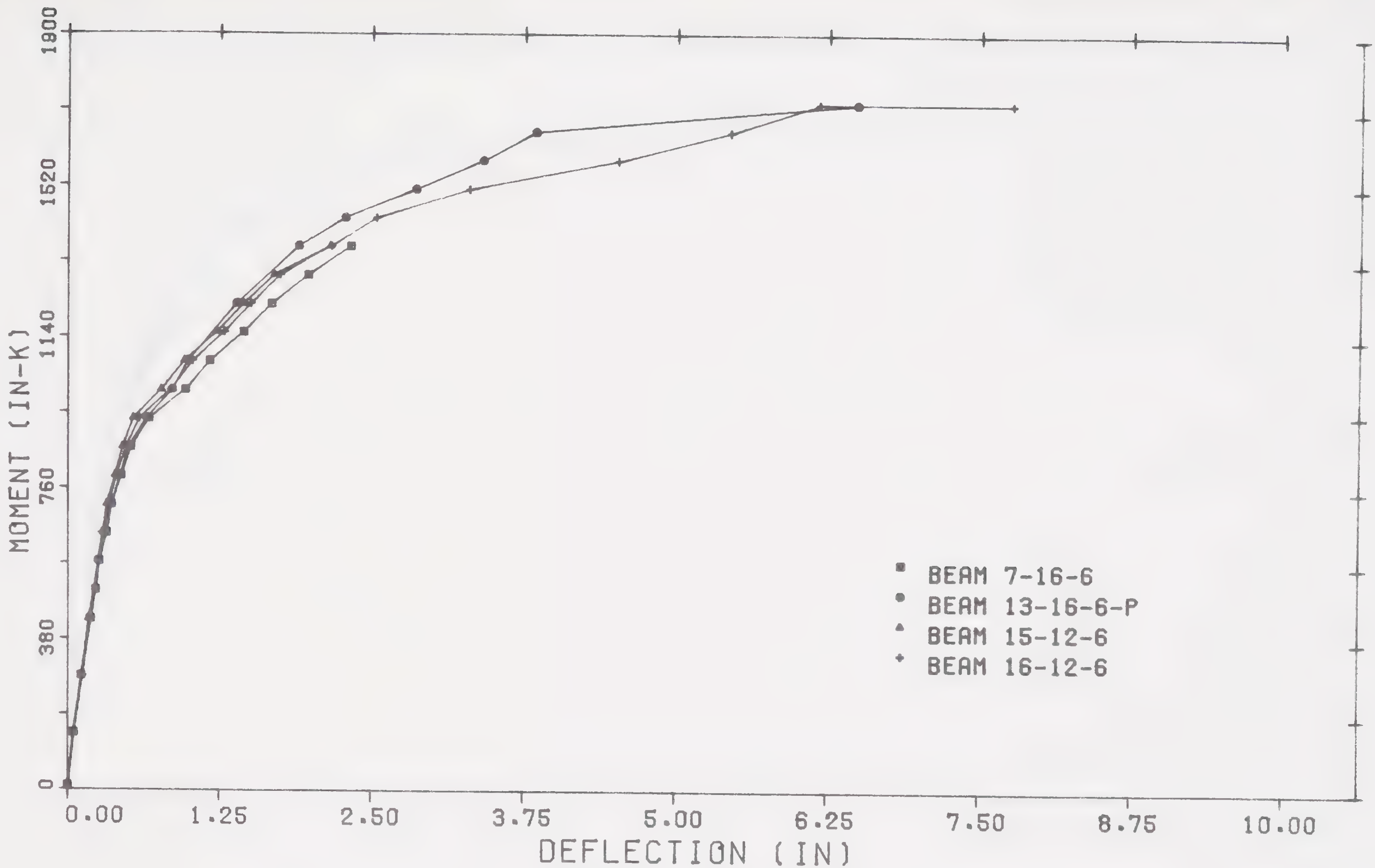


FIG. 4.7 MOMENT DEFLECTION FOR 6FT. SHEAR SPAN

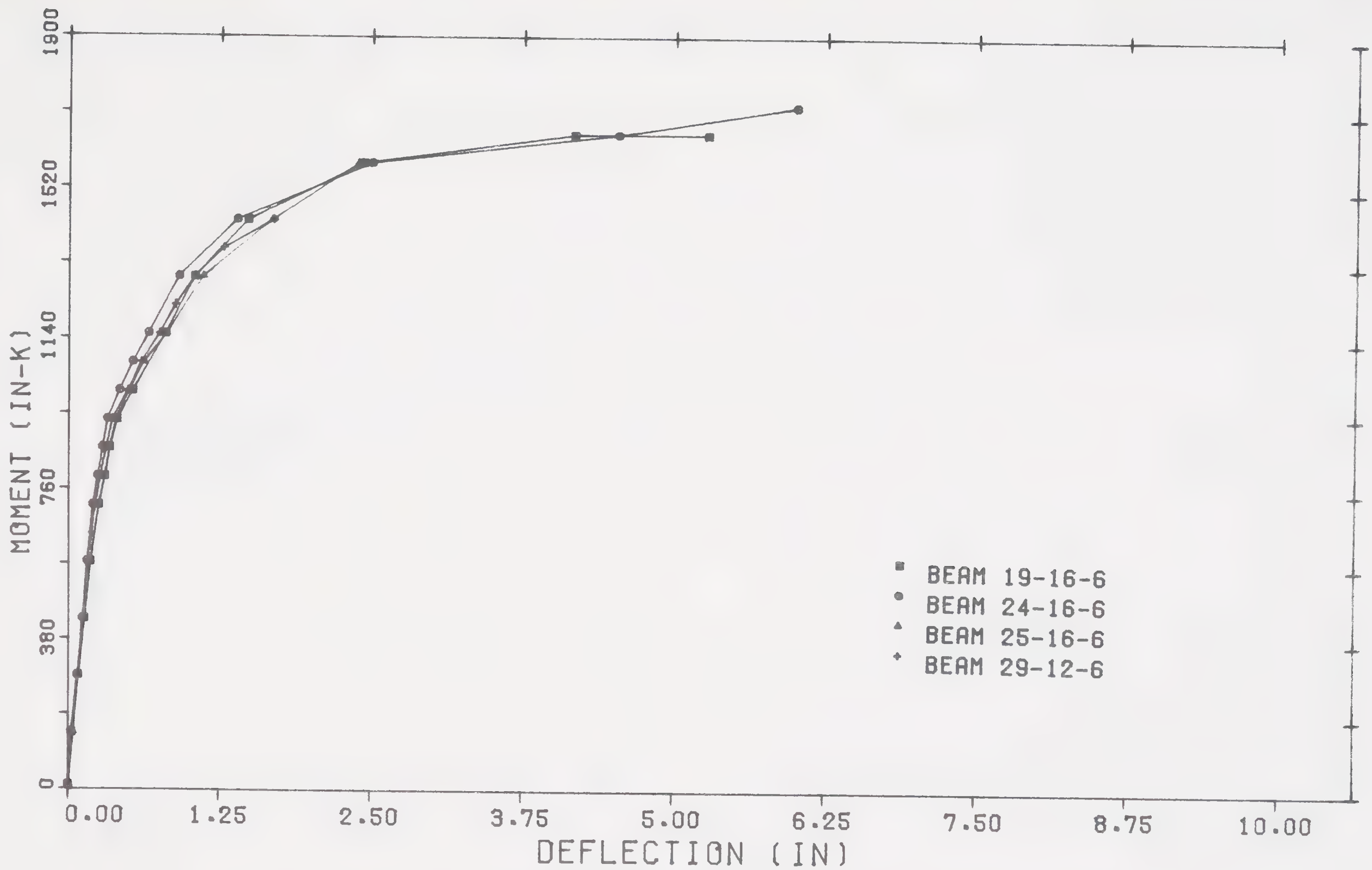


FIG. 4.8 MOMENT DEFLECTION FOR 6FT. SHEAR SPAN

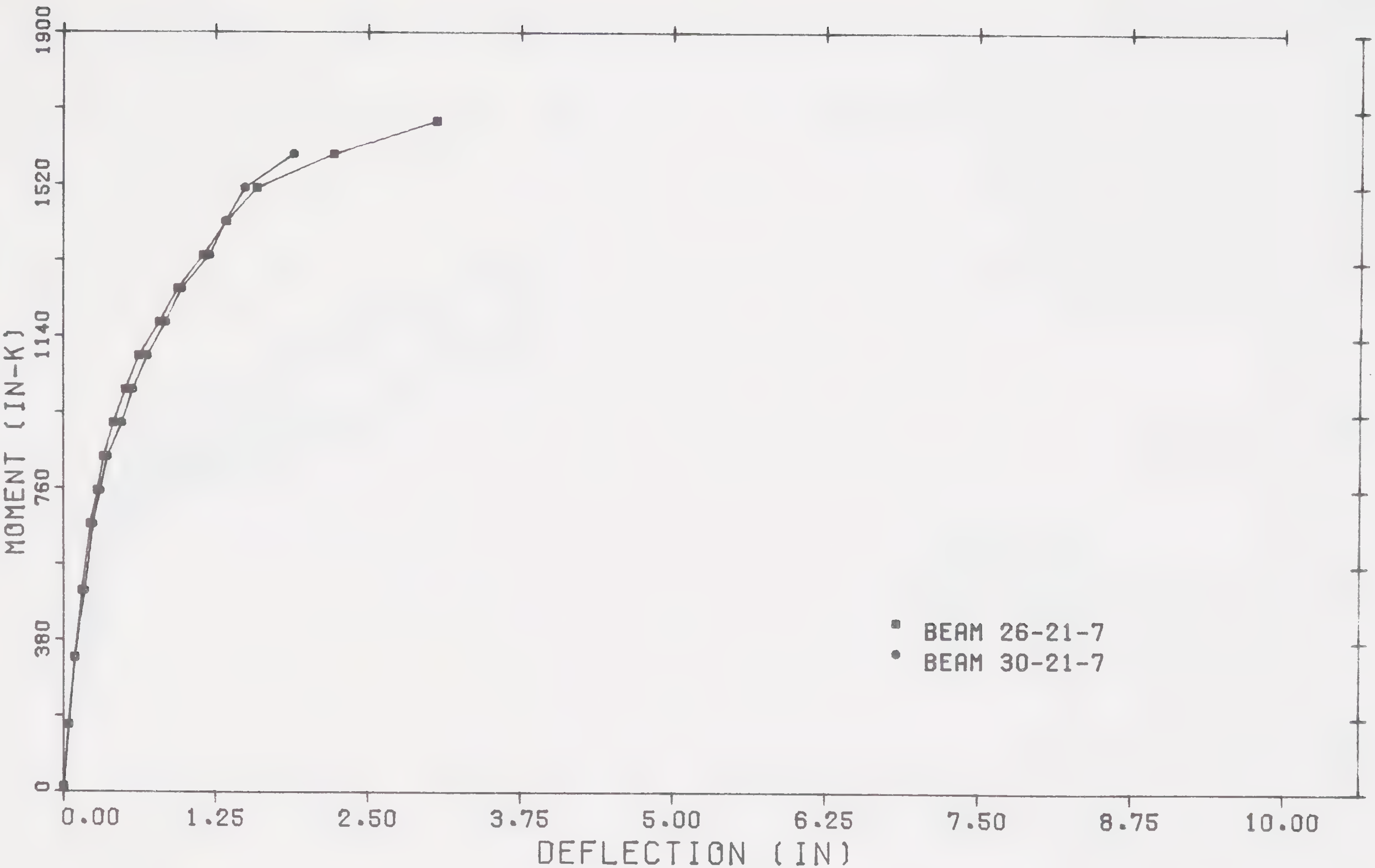


FIG. 4.9 MOMENT DEFLECTION FOR 7FT. SHEAR SPAN

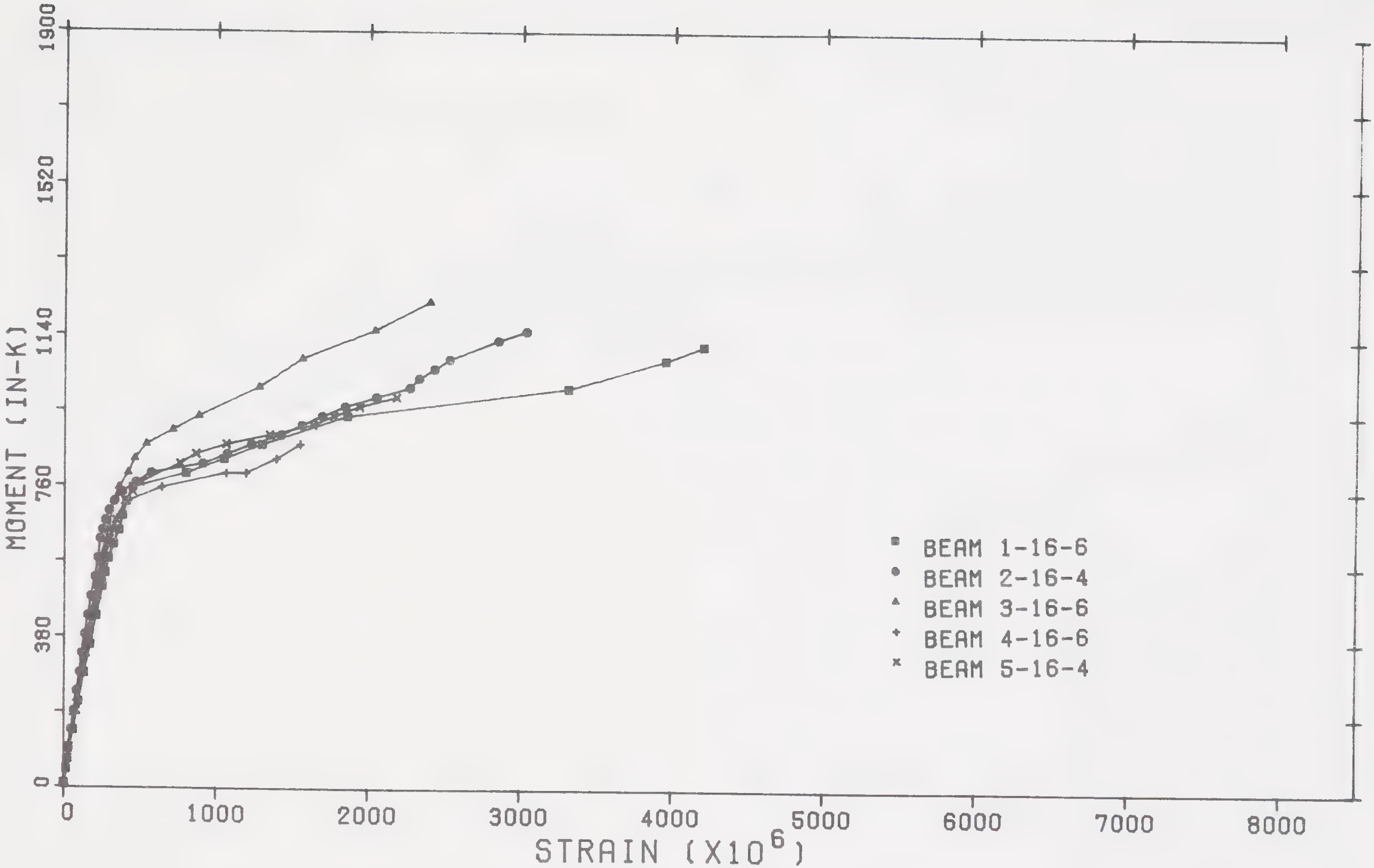


FIG. 4.10 MOMENT VS. STRAND STRAIN GAGE 1

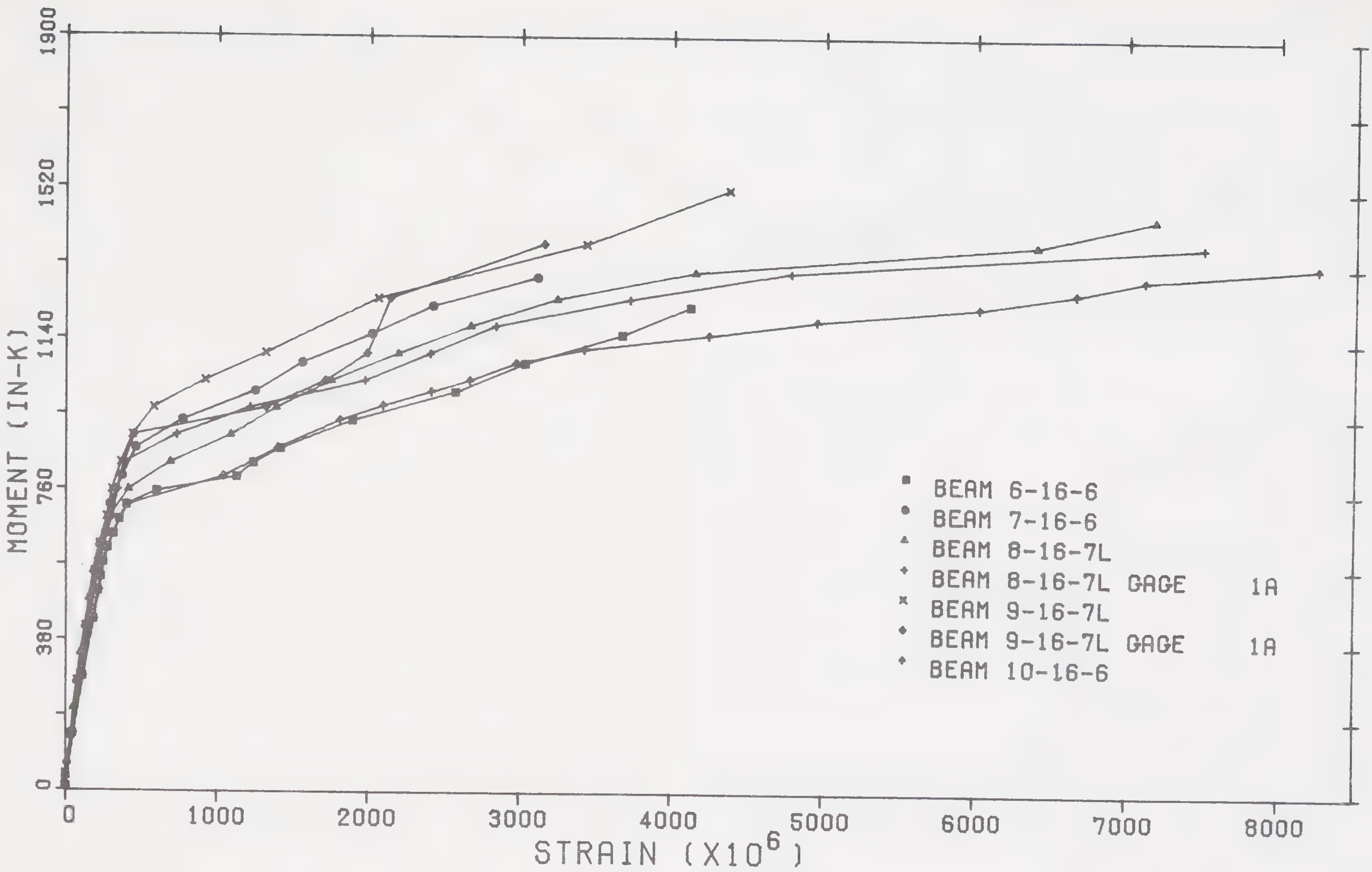


FIG. 4.11 MOMENT VS. STRAND STRAIN GAGE 1

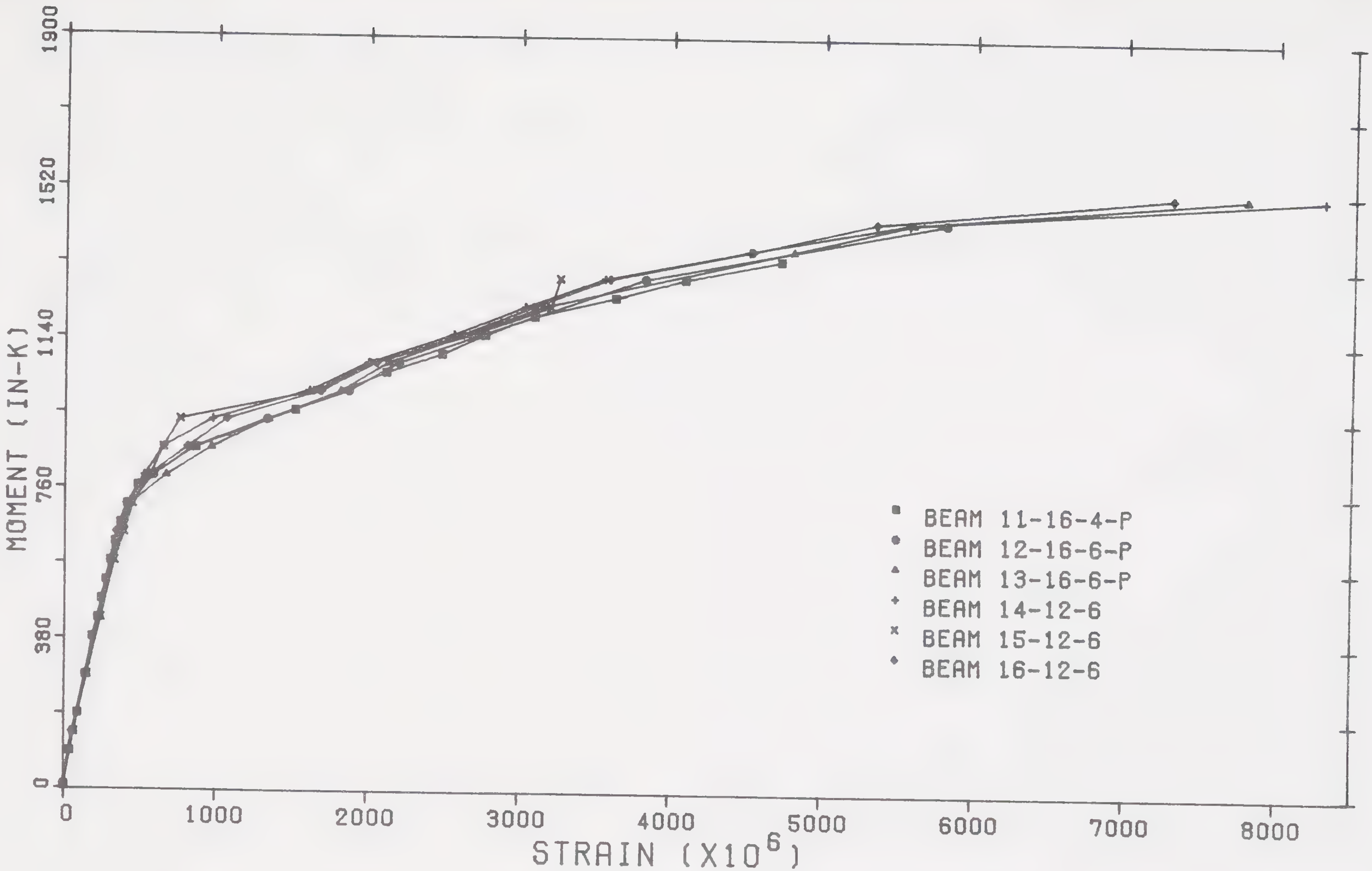


FIG. 4.12 MOMENT VS. STRAND STRAIN GAGE 1

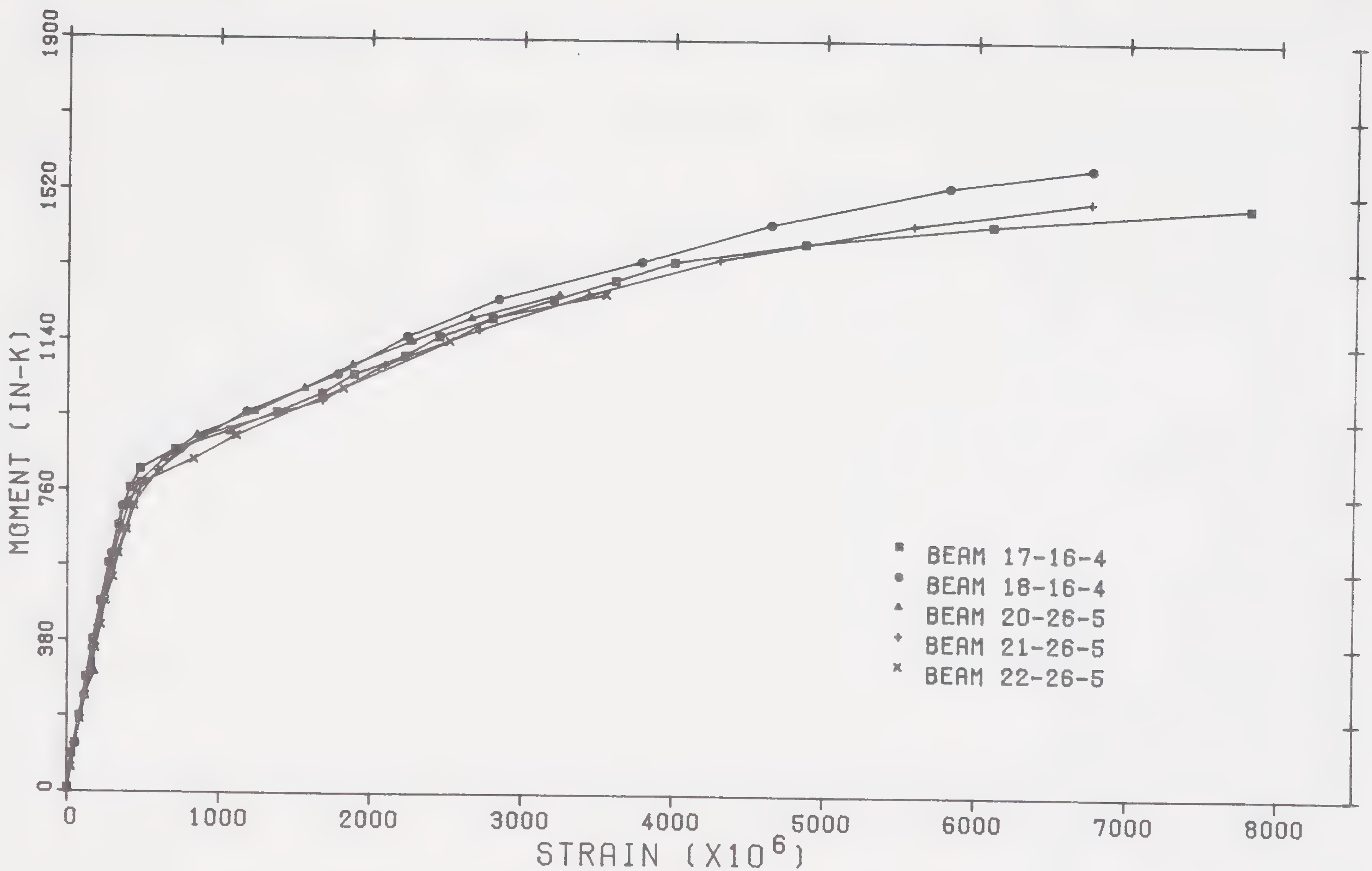


FIG. 4.13 MOMENT VS. STRAND STRAIN GAGE 1

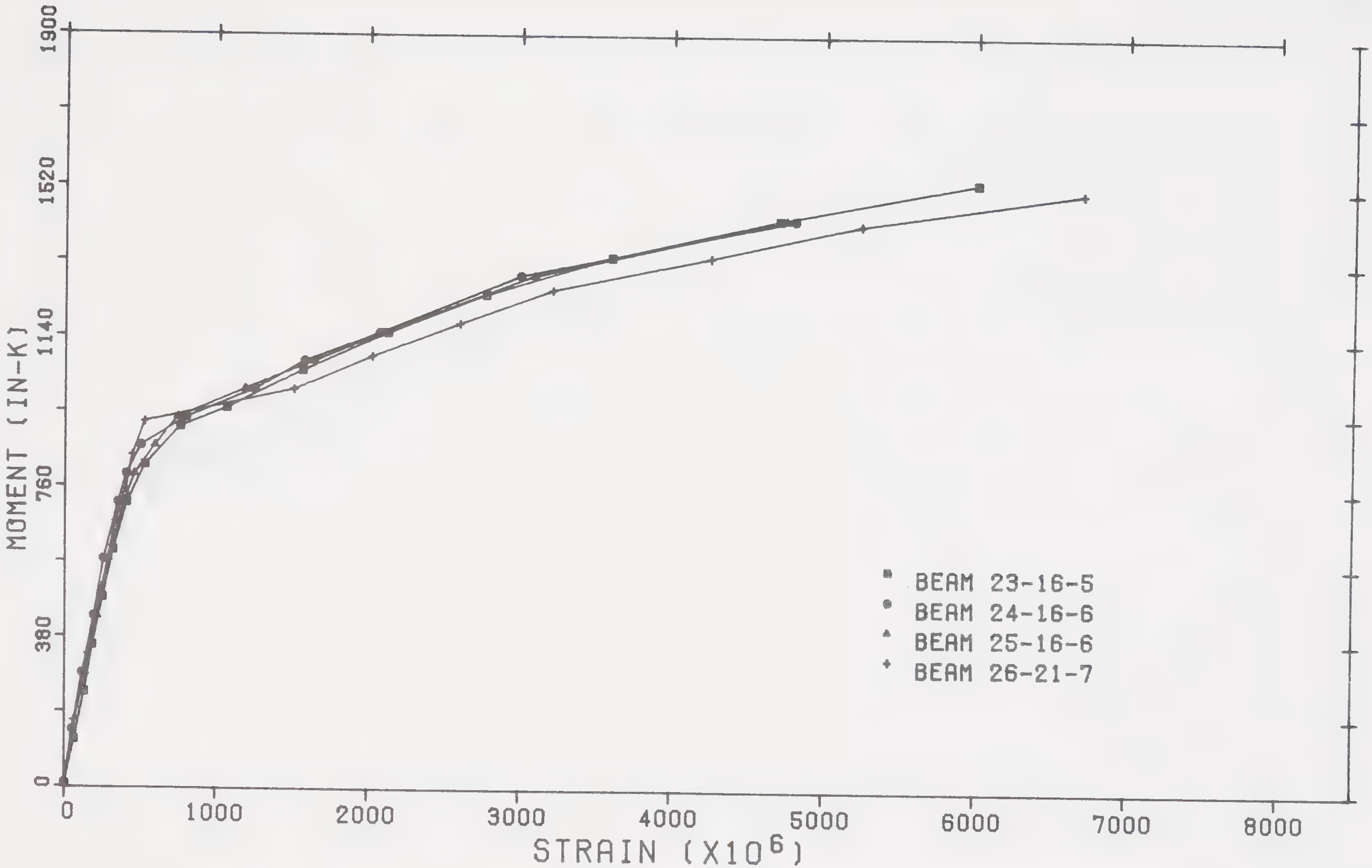


FIG. 4.14 MOMENT VS. STRAND STRAIN GAGE 1

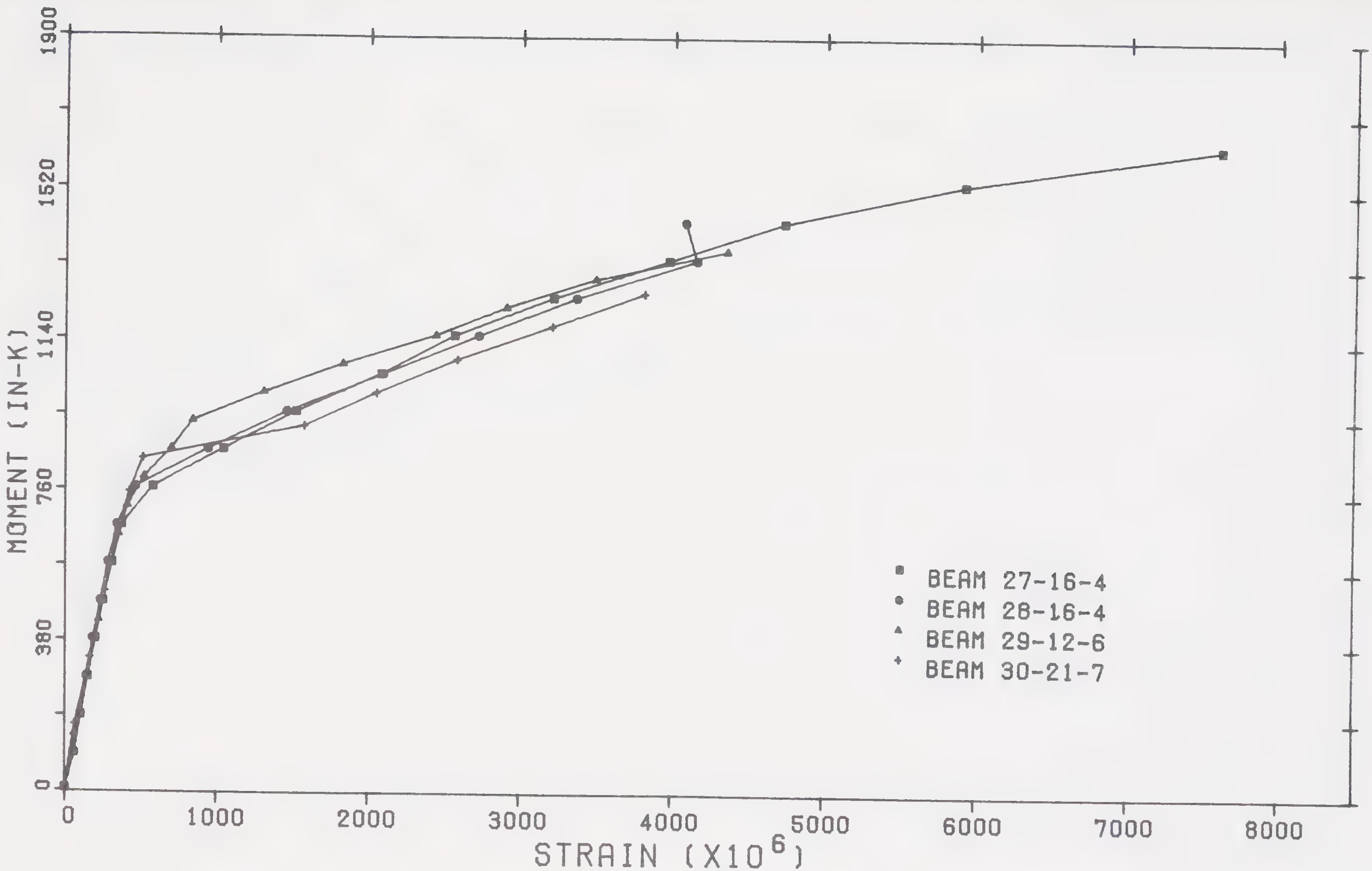


FIG. 4.15 MOMENT VS. STRAND STRAIN GAGE 1

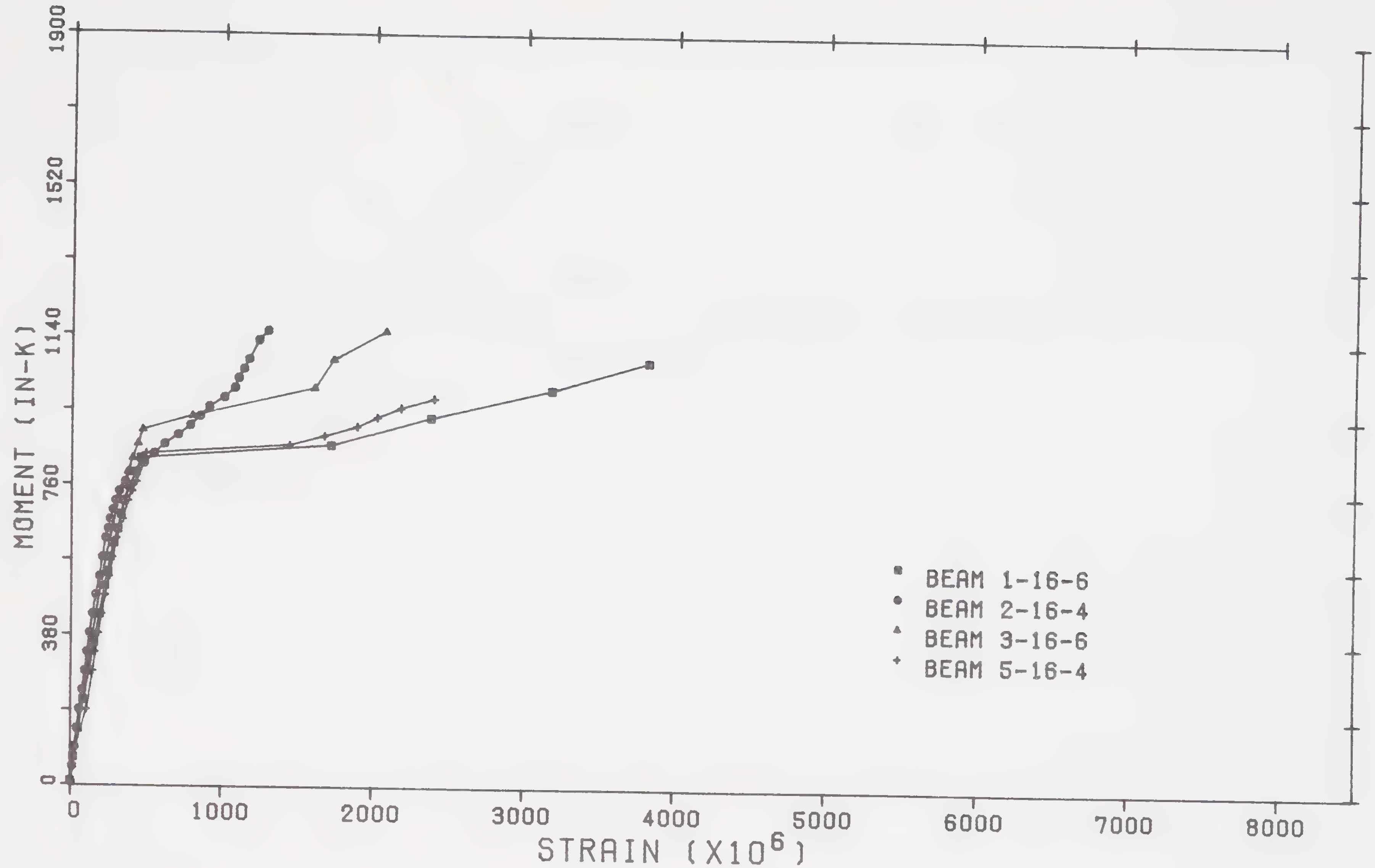


FIG. 4.16 MOMENT VS. STRAND STRAIN GAGE 2

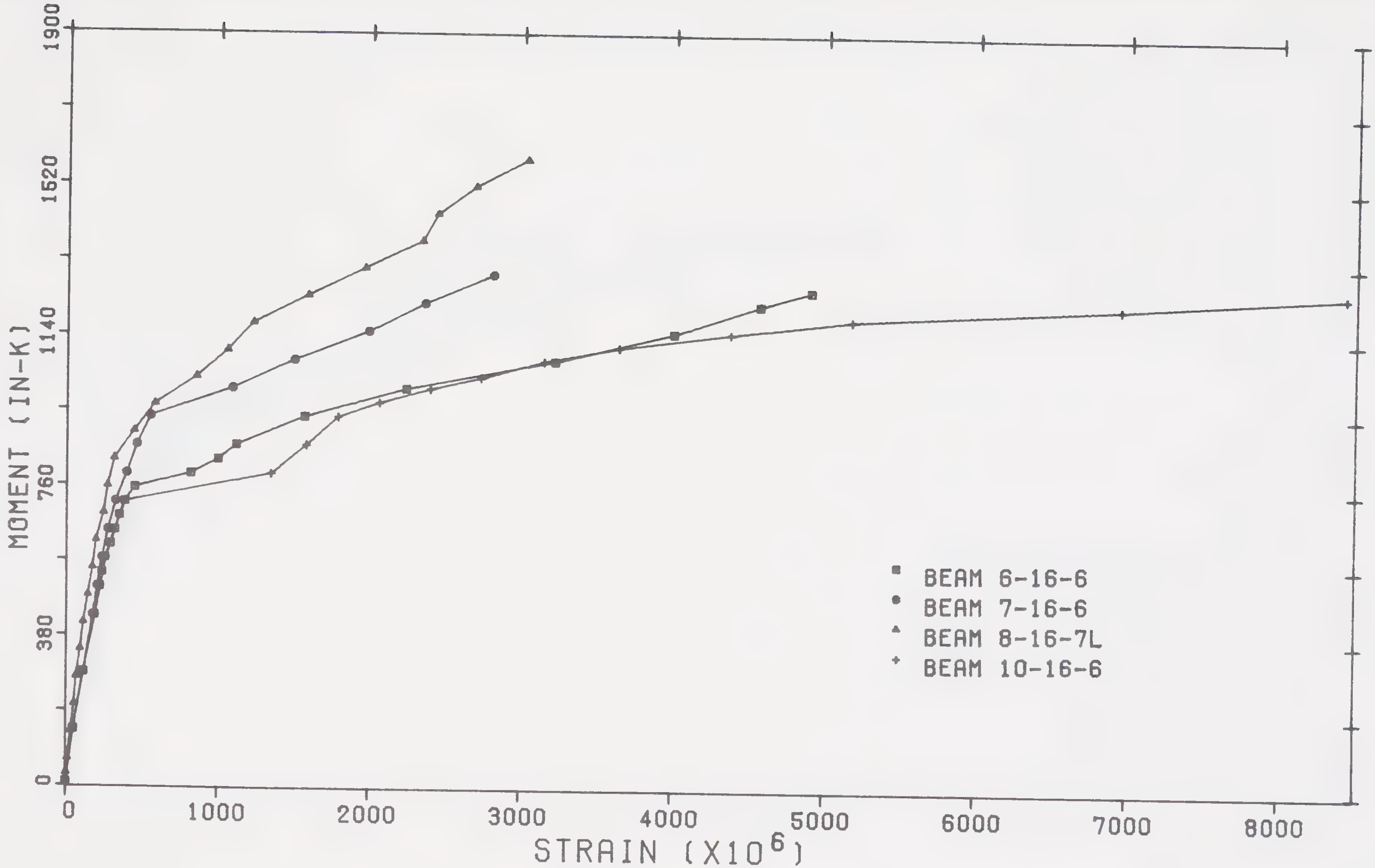


FIG. 4.17 MOMENT VS. STRAND STRAIN GAGE 2

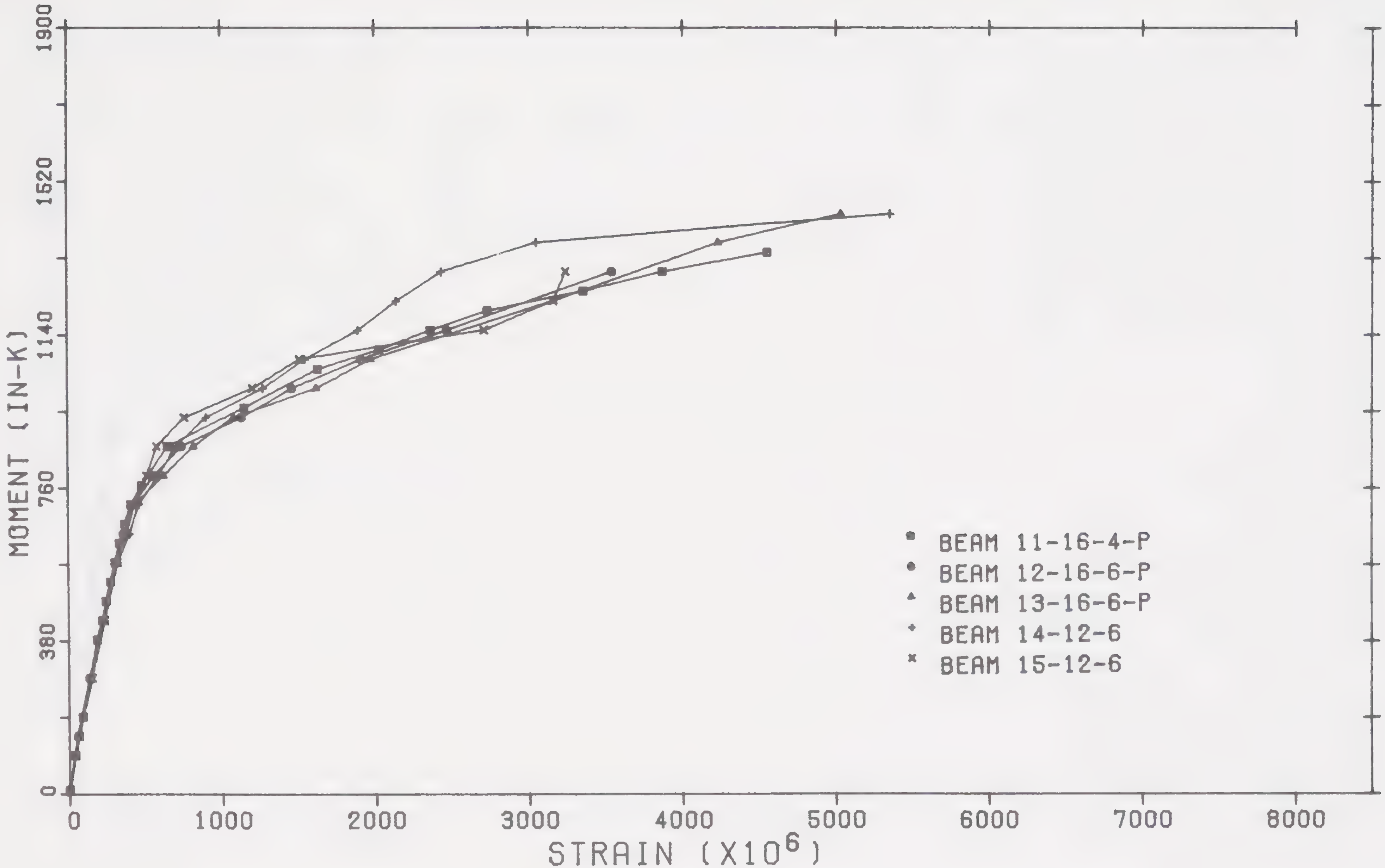


FIG. 4.18 MOMENT VS. STRAND STRAIN GAGE 2

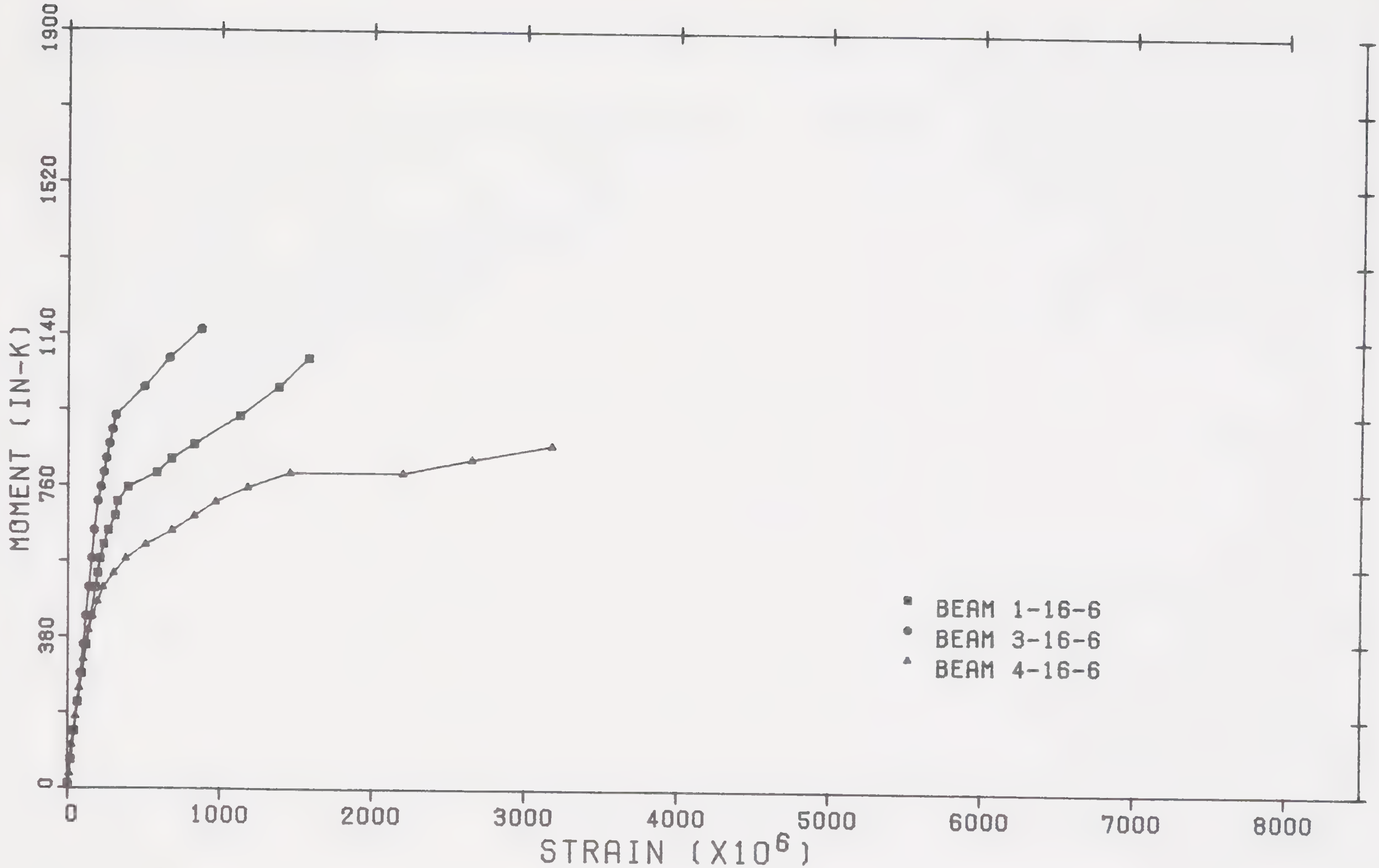


FIG. 4.19 MOMENT VS. STRAND STRAIN GAGE 3

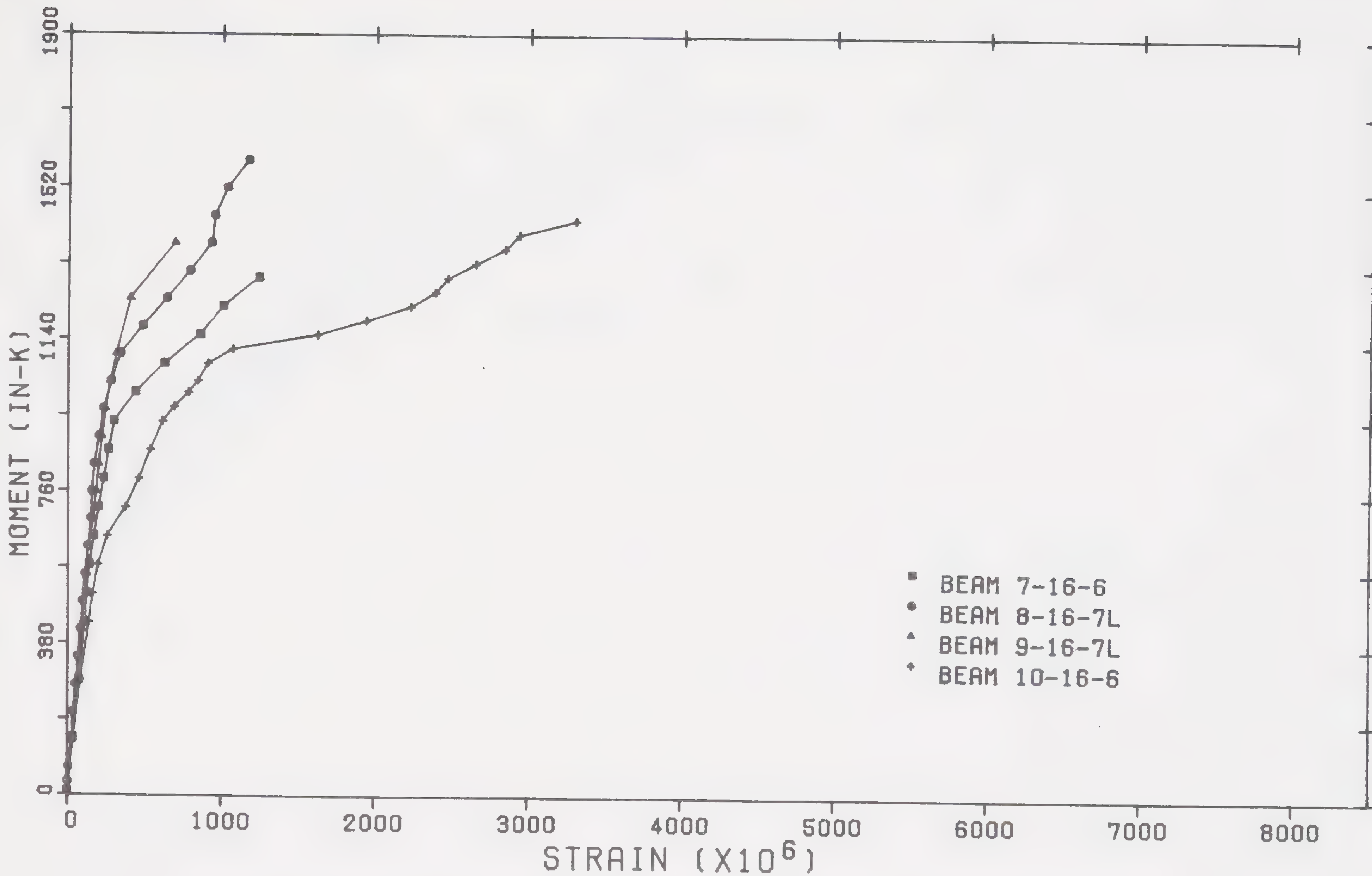


FIG. 4.20 MOMENT VS. STRAND STRAIN GAGE 3

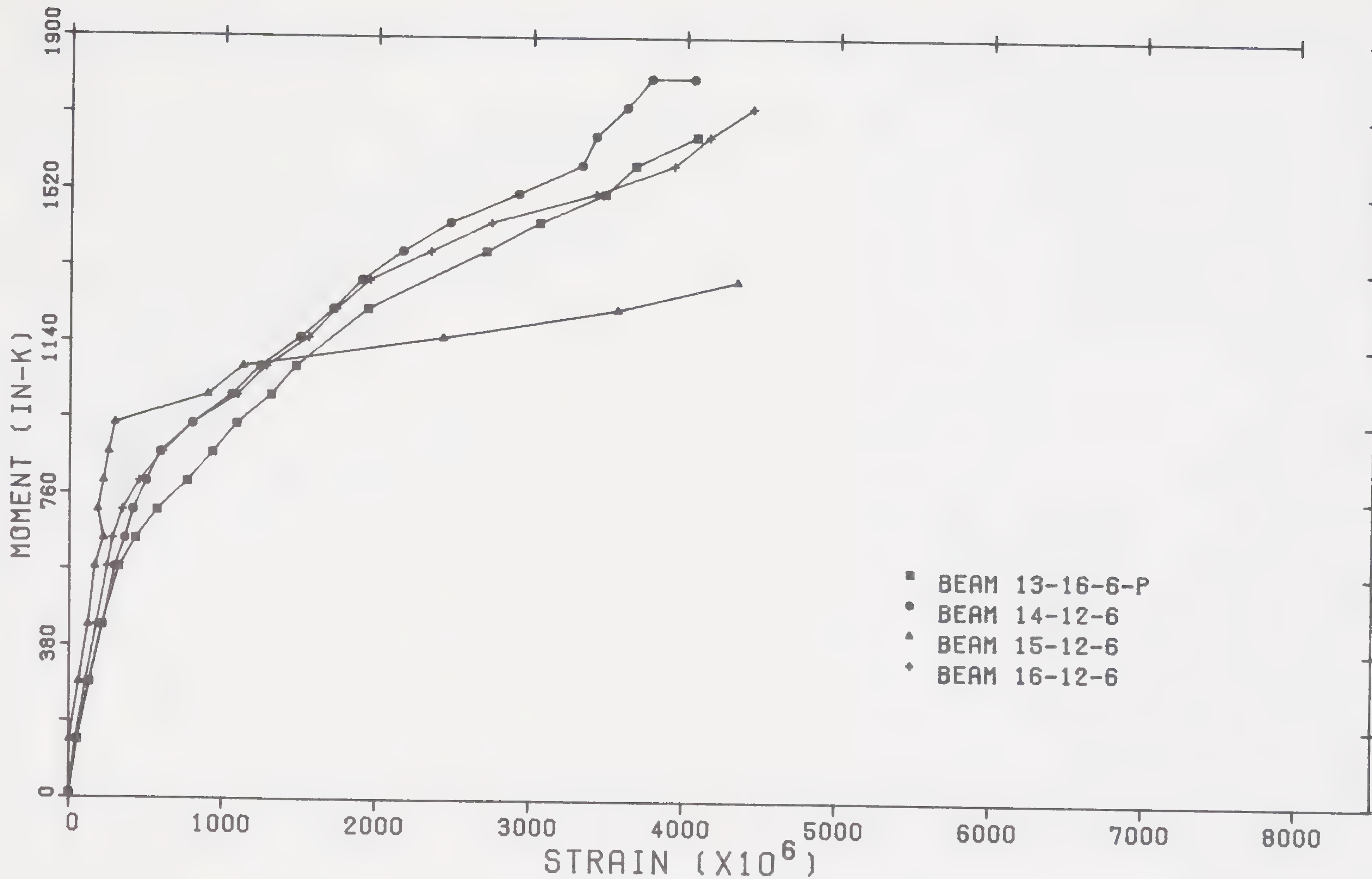


FIG. 4.21 MOMENT VS. STRAND STRAIN GAGE 3

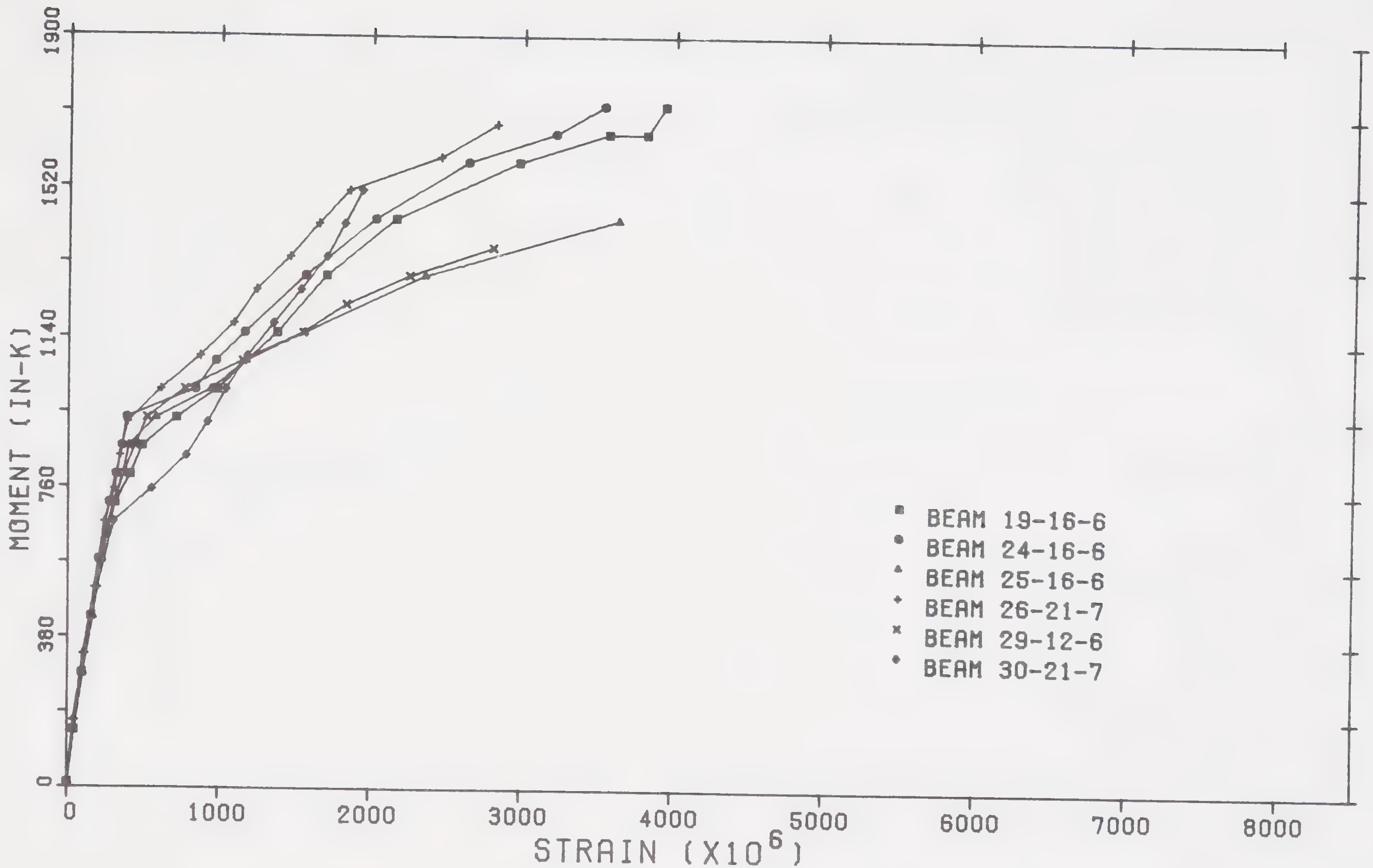


FIG. 4.22 MOMENT VS. STRAND STRAIN GAGE 3

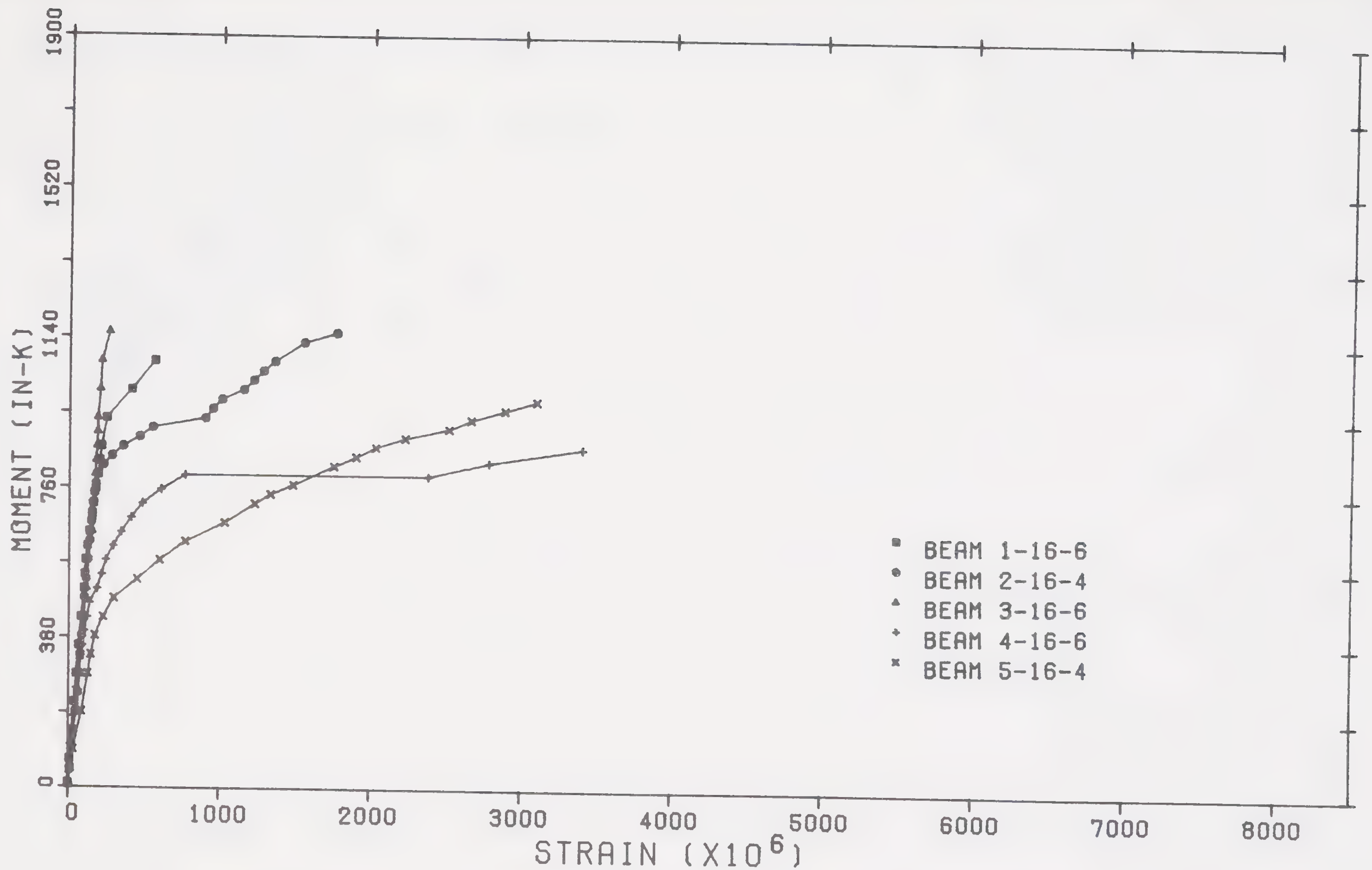


FIG. 4.23 MOMENT VS. STRAND STRAIN GAGE 4

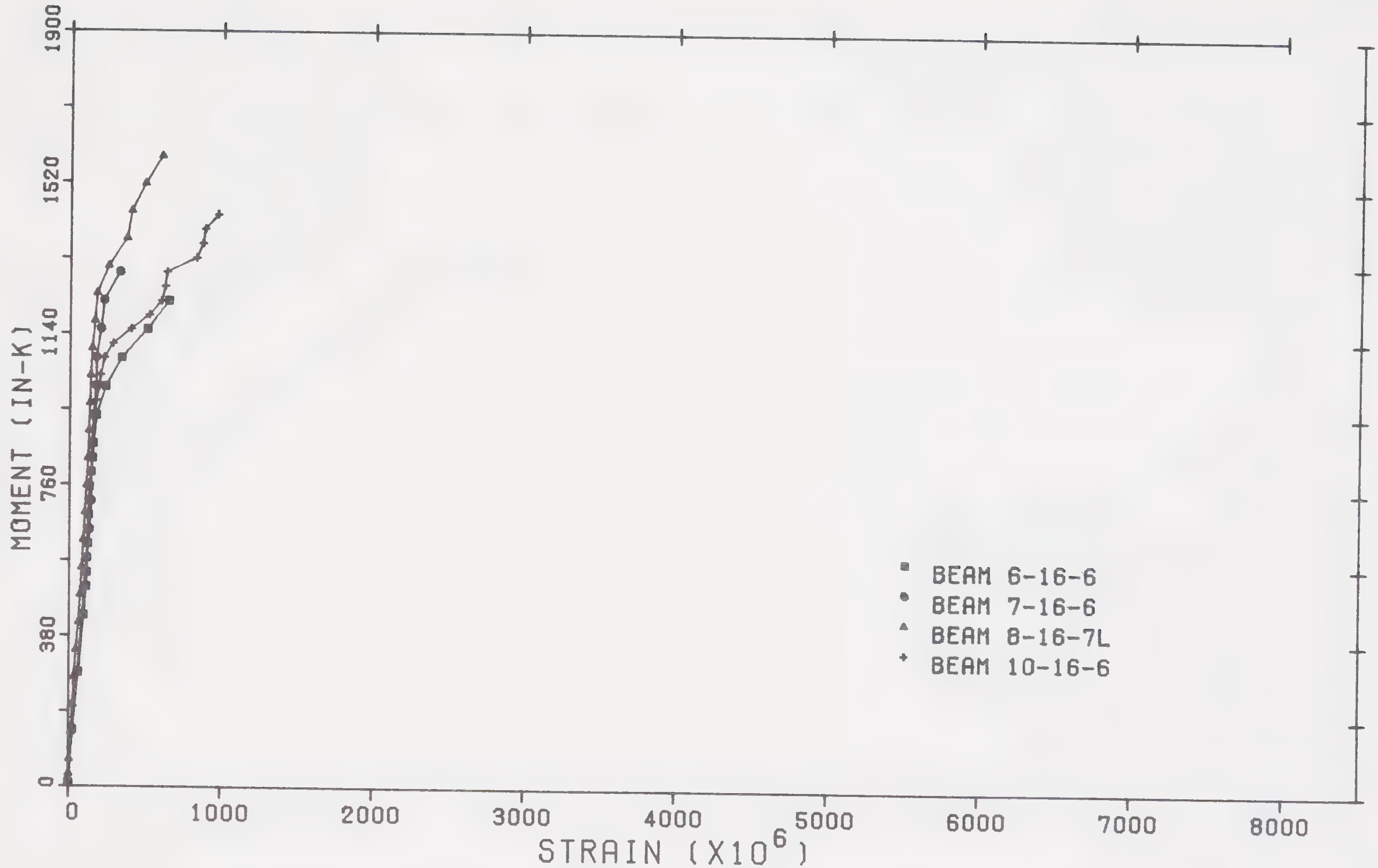


FIG. 4.24 MOMENT VS. STRAND STRAIN GAGE 4

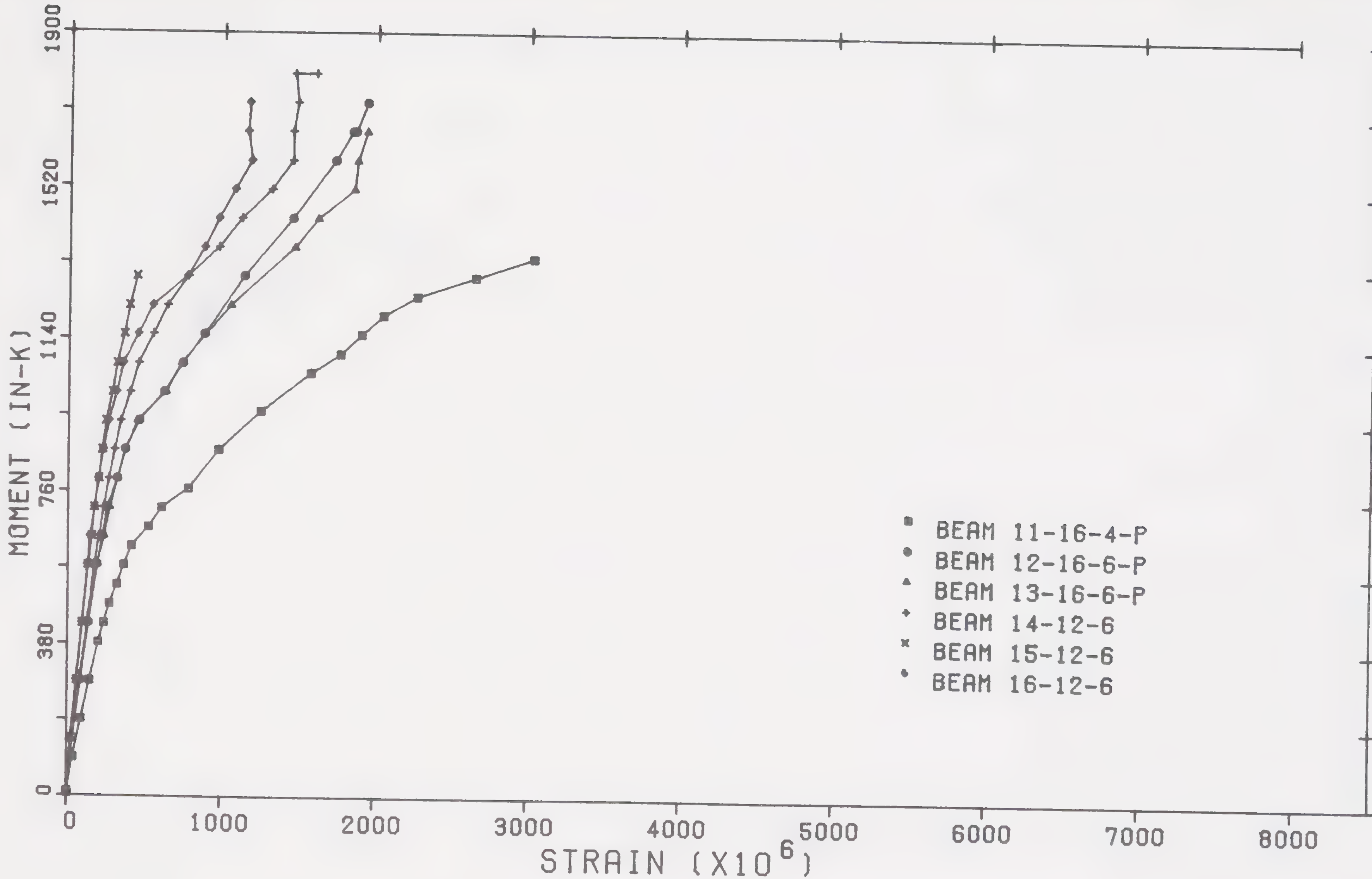


FIG. 4.25 MOMENT VS. STRAND STRAIN GAGE 4

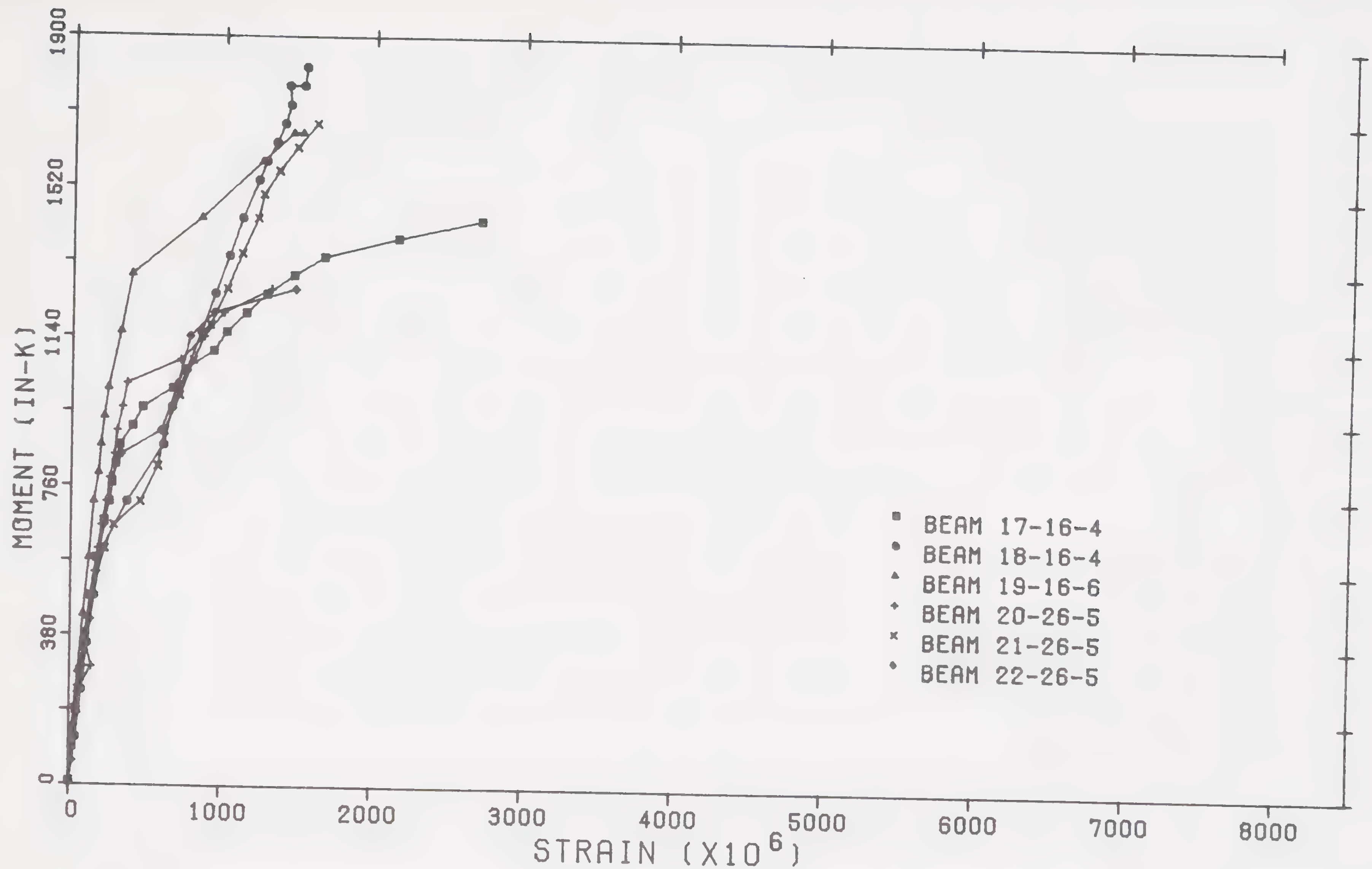


FIG. 4.26 MOMENT VS. STRAND STRAIN GAGE 4

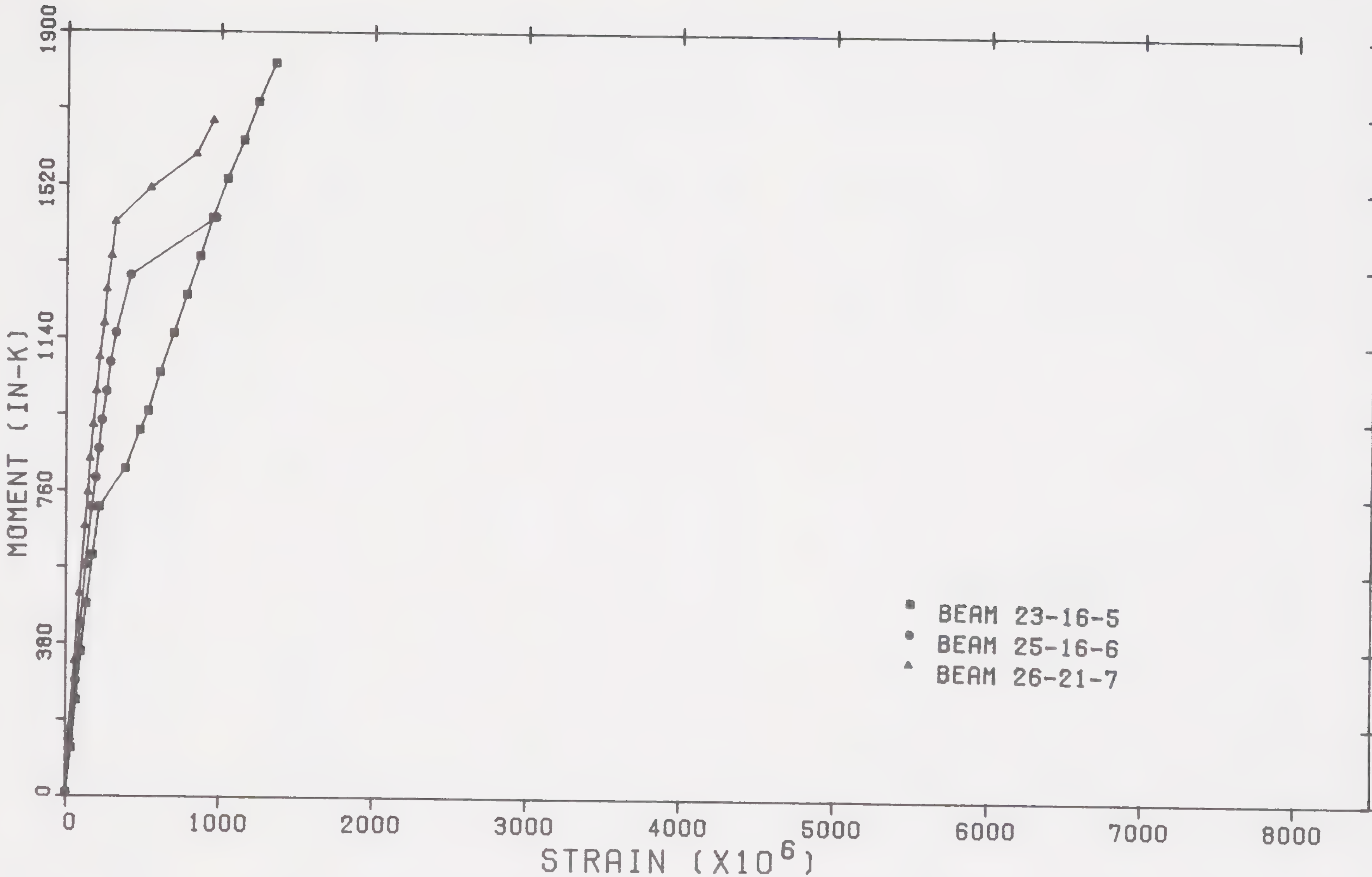


FIG. 4.27 MOMENT VS. STRAND STRAIN GAGE 4

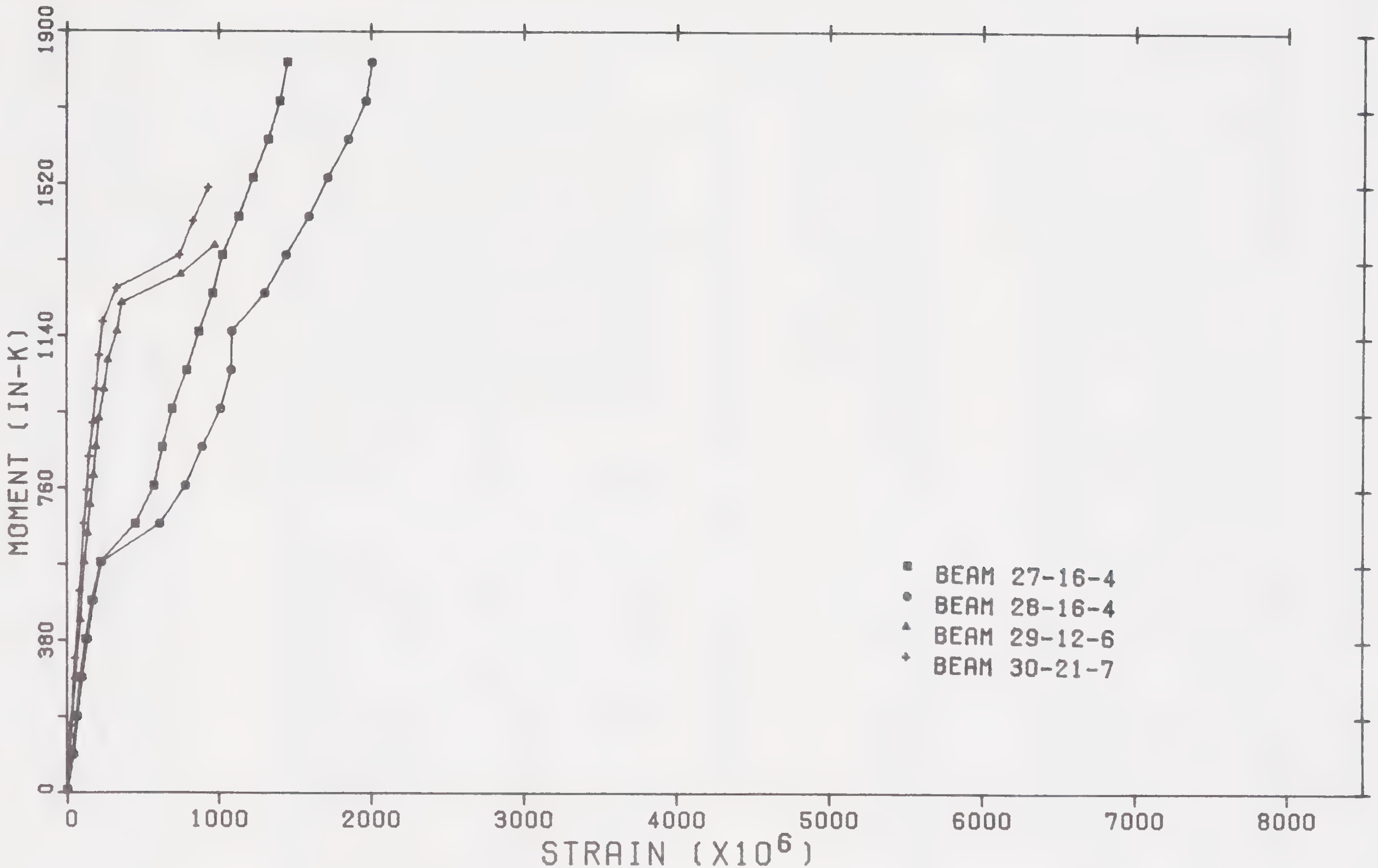


FIG. 4.28 MOMENT VS. STRAND STRAIN GAGE 4

The plotted results are grouped for each gage location and the curves are plotted in order of the beam tests. The gage location, loads and strain readings are given in Appendix B.

The total strain in the prestressing strand can be obtained by adding the strain due to the effective prestressing force and the plotted strain.

One hundred and one gages were mounted on the prestressing strand for all beams; 7% of these were inoperative at the time of testing and are not plotted.

4.5 Load Strain Relationships for the Shear Reinforcement

Load versus strain in the shear reinforcement is plotted in Figures 4.29 to 4.88 for gage locations 5 to 24, 41 and 42. The strains are those read directly from the electrical resistance strain gages and the loads are the loads per jack read from the scales of the Amsler loading apparatus. The plots are grouped in numerical order of the gage loactions and the curves are in numerical order of the beams tested. The general gage locations are shown in Figure 3.5 and the gage locations for each beam are given in Appendix B together with the tabulated data for the curves.

The strain in the full length stirrups in the posts and solid shear spans at gage locations 5 to 8 are plotted in Figures 4.29 to 4.47. The strain in the top strut stirrups at gage locations 9 to 16

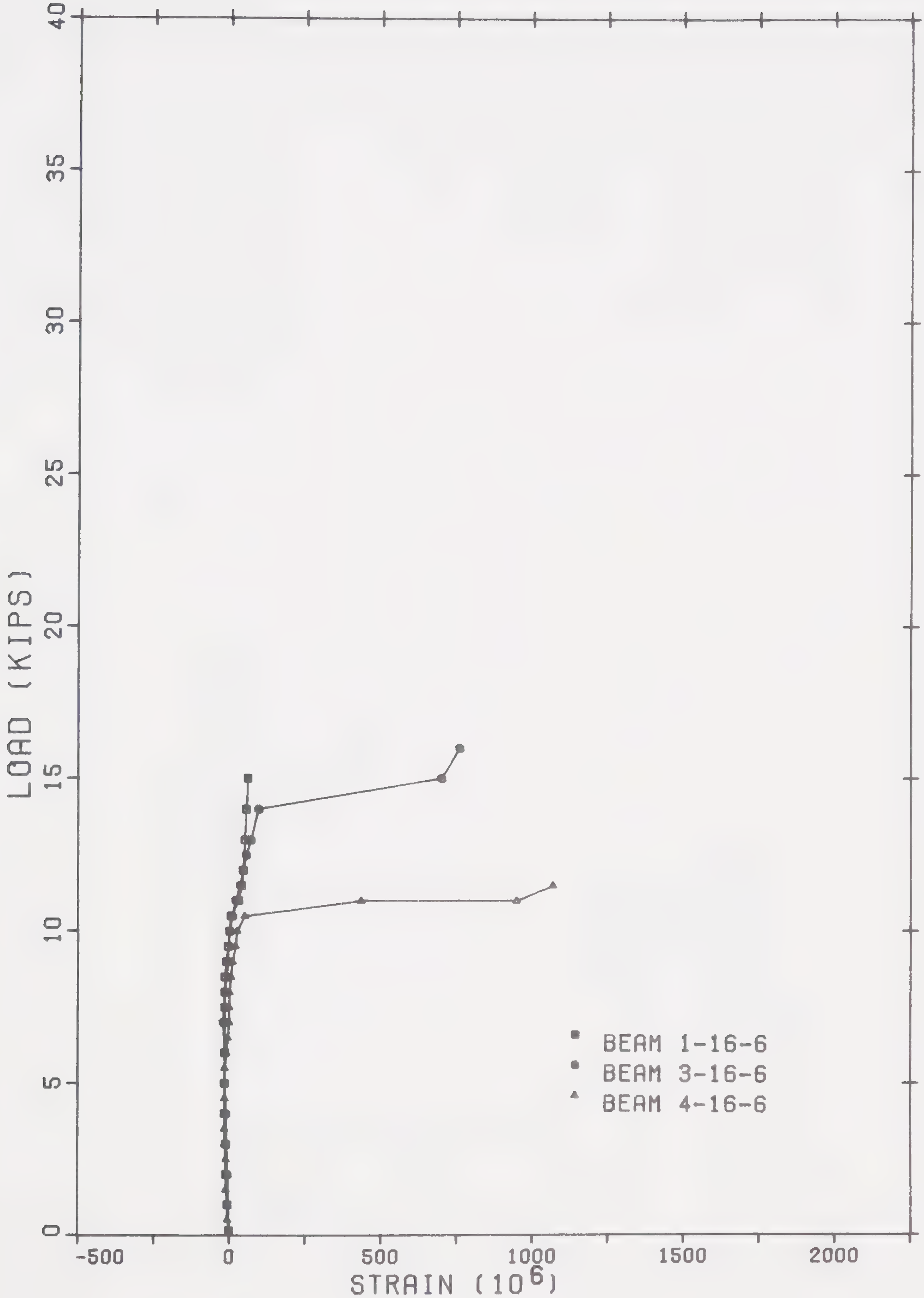


FIG. 4.29 LOAD VS. STRAIN GAGE 5

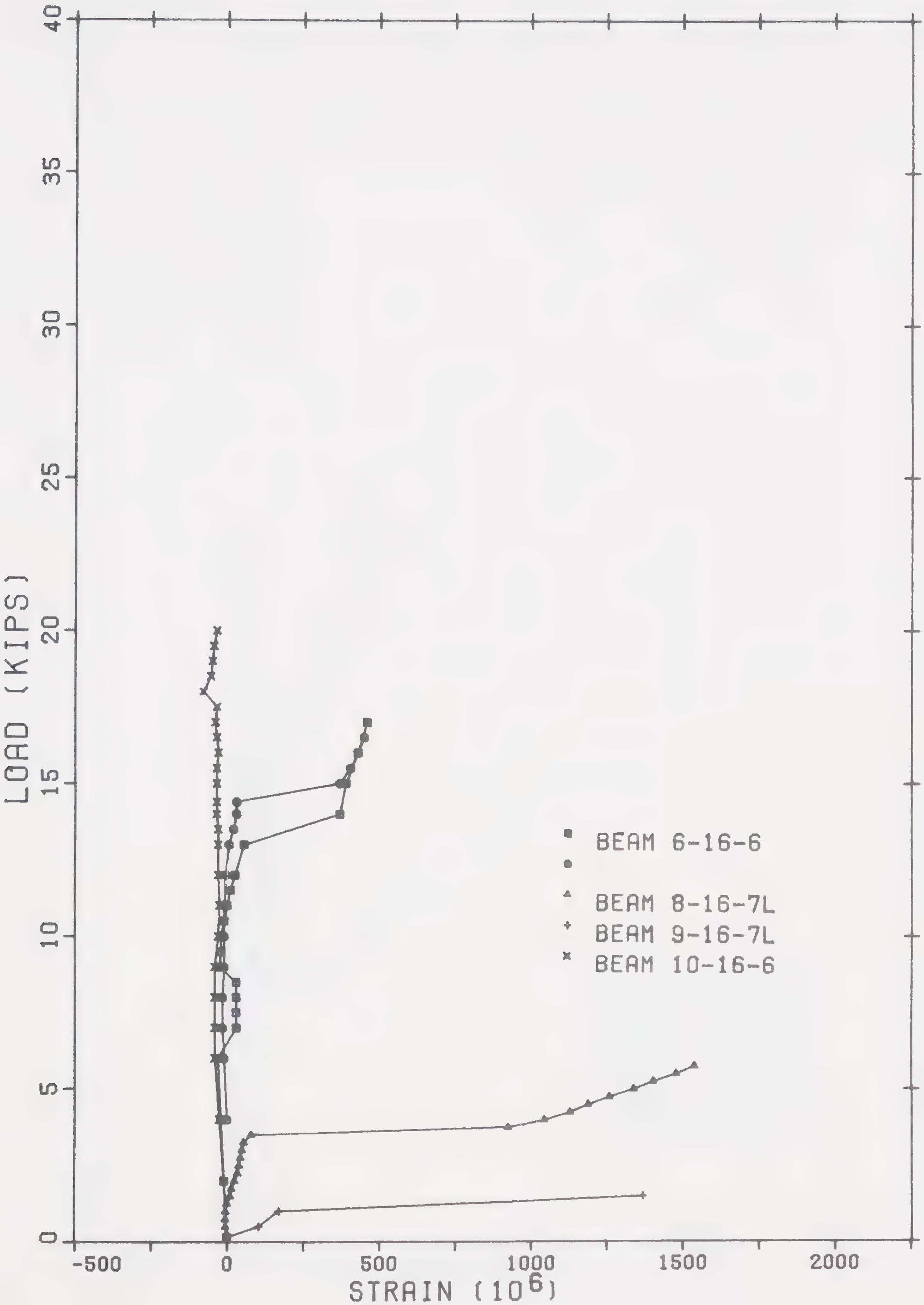


FIG. 4.30 LOAD VS. STRAIN GAGE 5

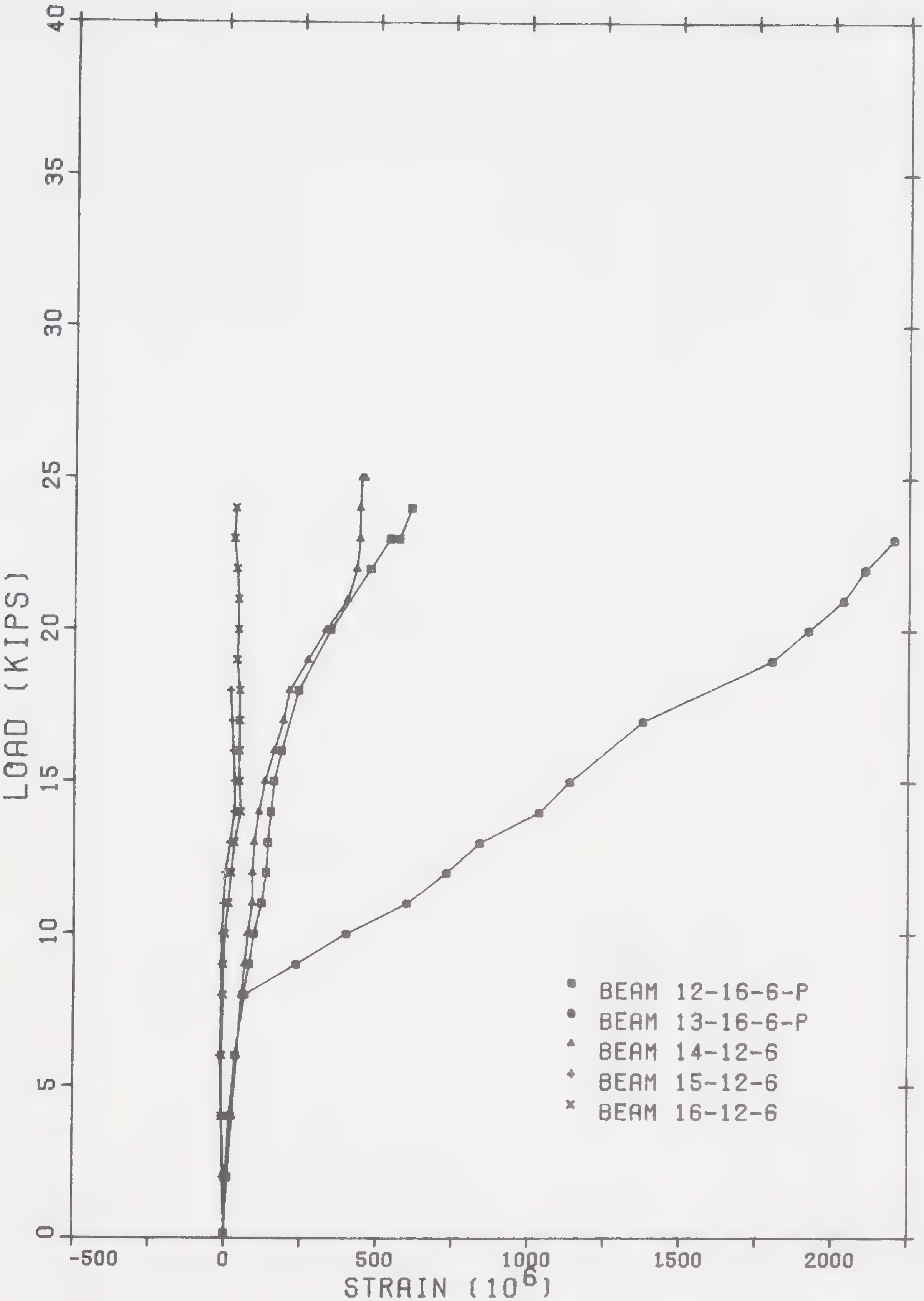


FIG. 4.31 LOAD VS. STRAIN GAGE 5

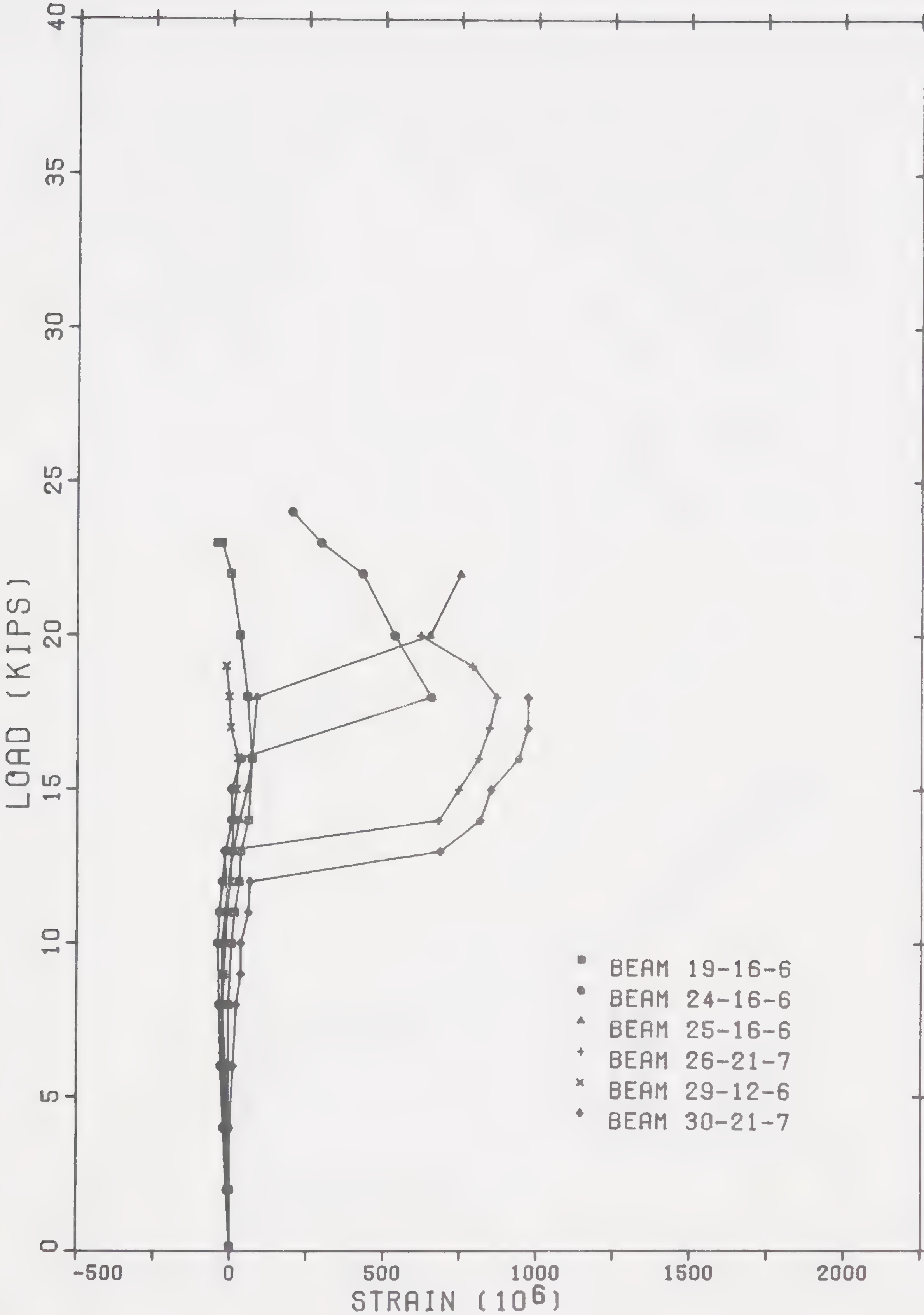


FIG. 4.32 LOAD VS. STRAIN GAGE 5

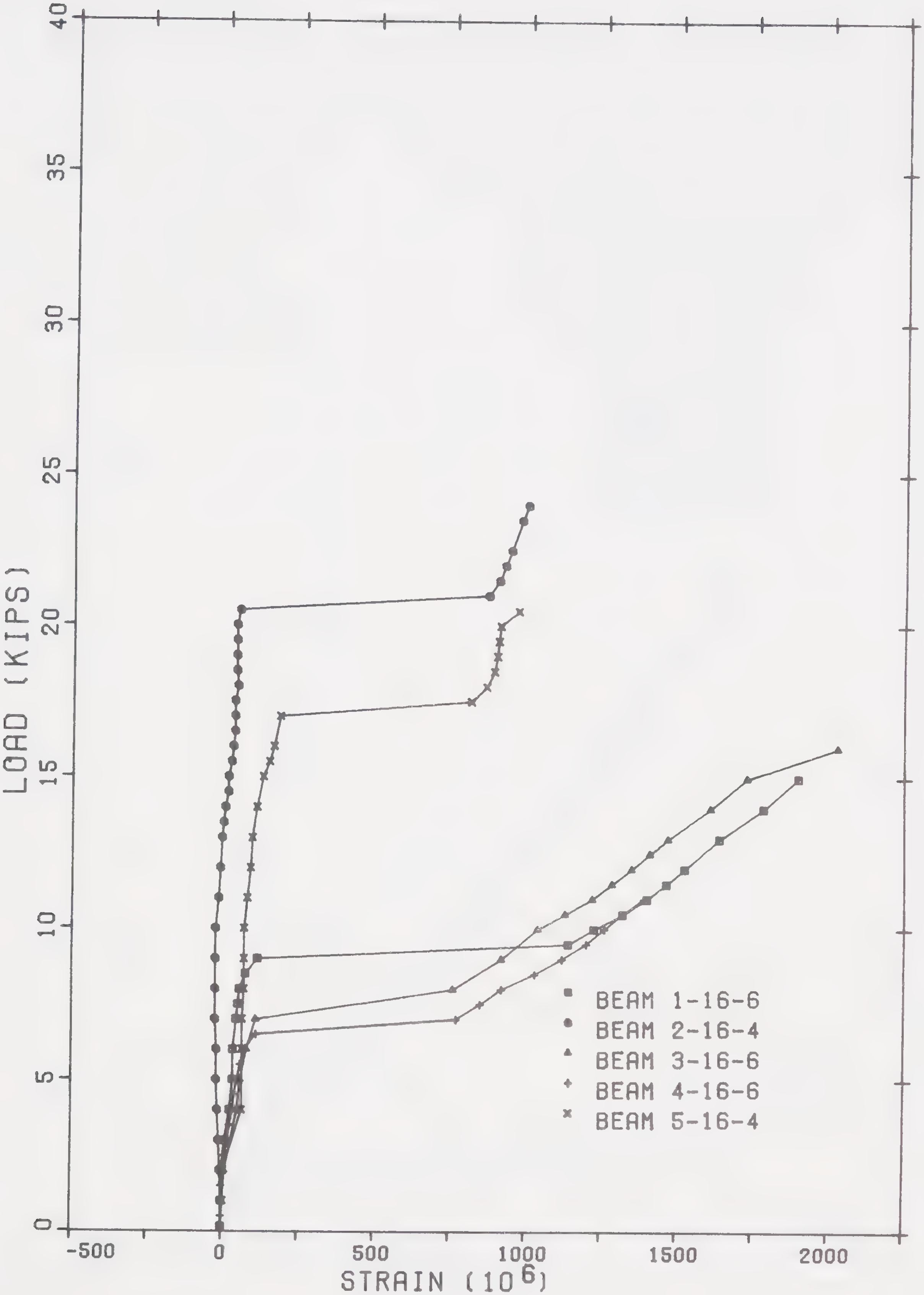


FIG. 4.33 LOAD VS. STRAIN GAGE 6

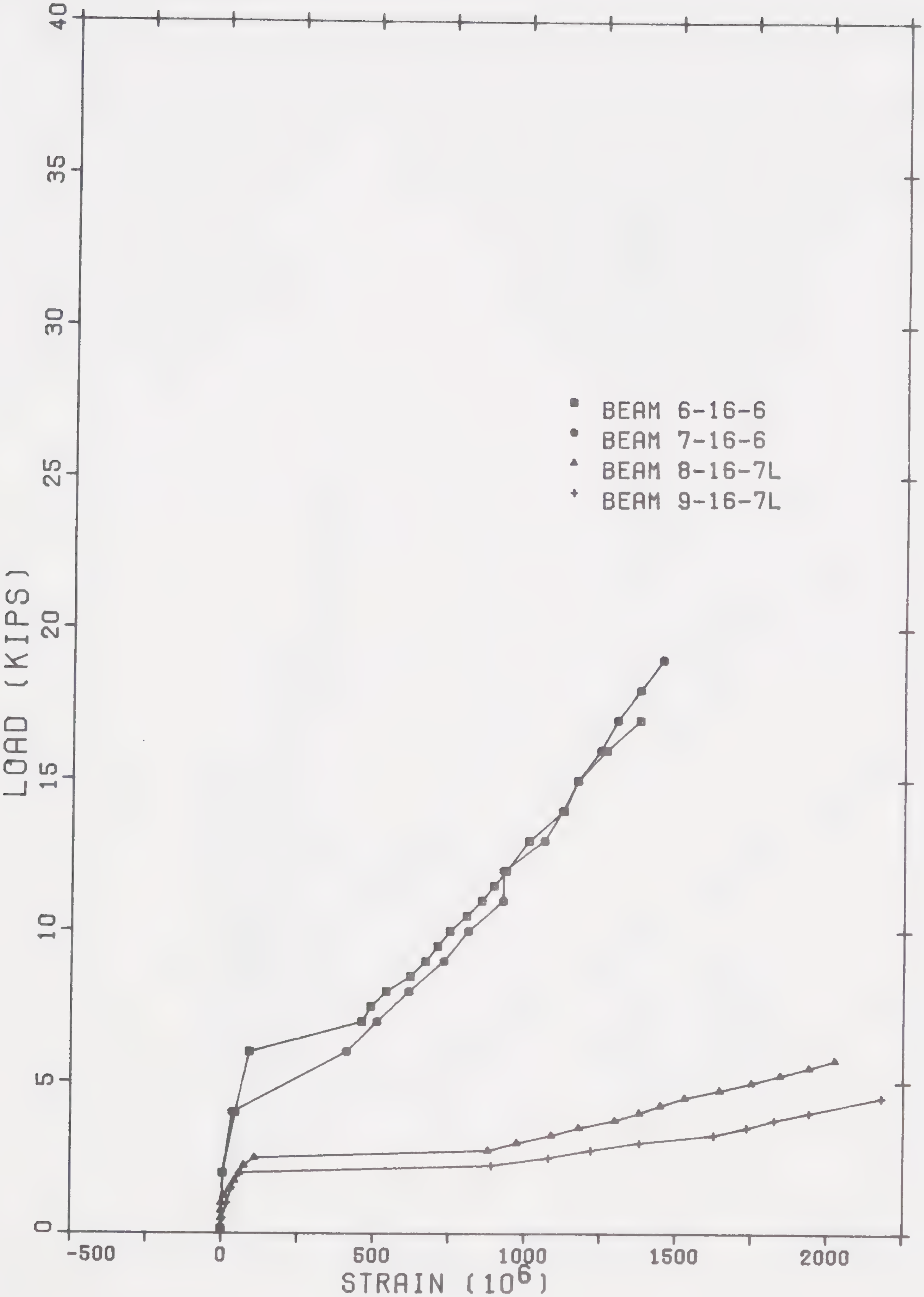


FIG. 4.34 LOAD VS. STRAIN GAGE 6

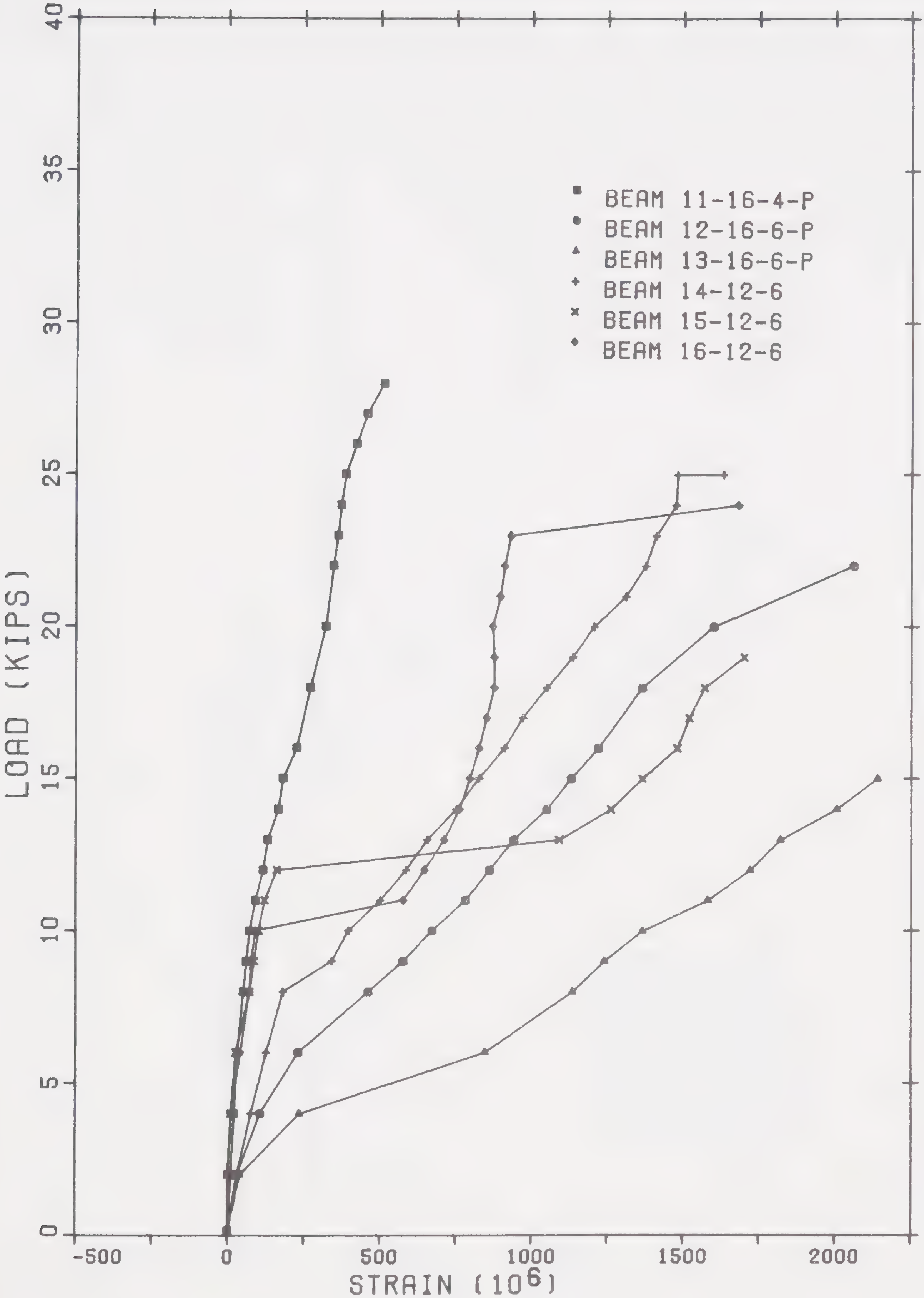


FIG. 4.35 LOAD VS. STRAIN GAGE 6

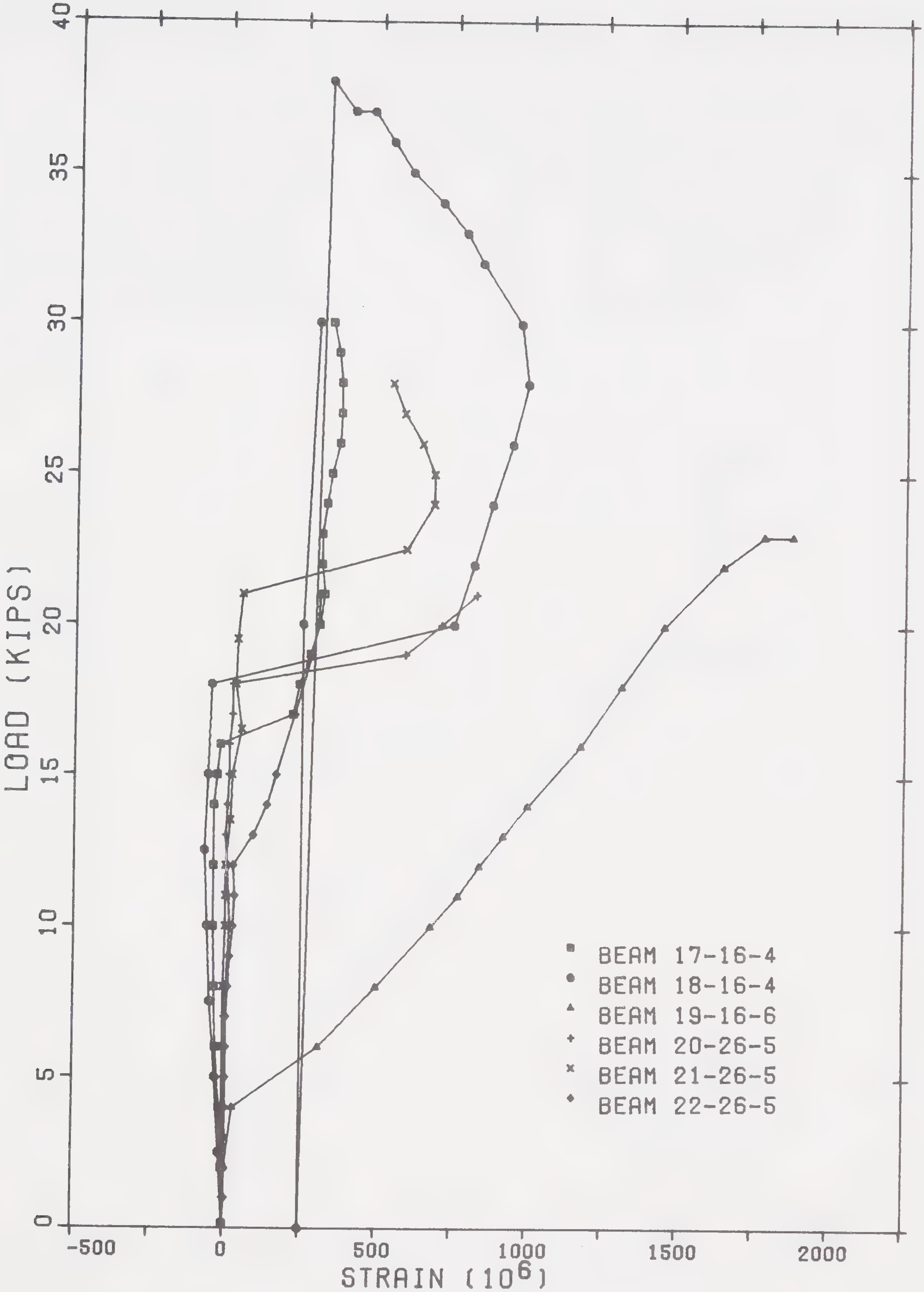


FIG. 4.36 LOAD VS. STRAIN GAGE 6

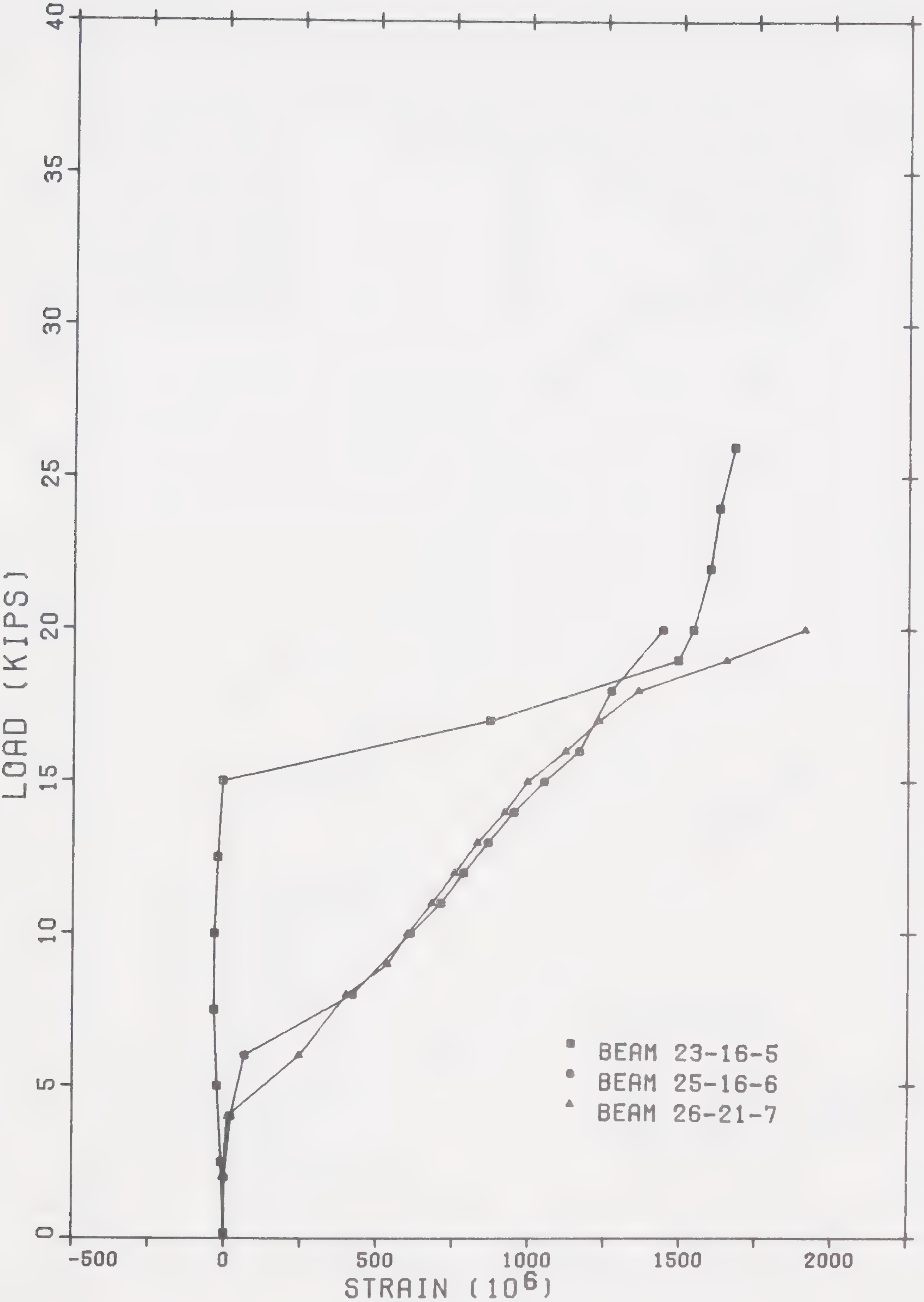


FIG. 4.37 LOAD VS. STRAIN GAGE 6

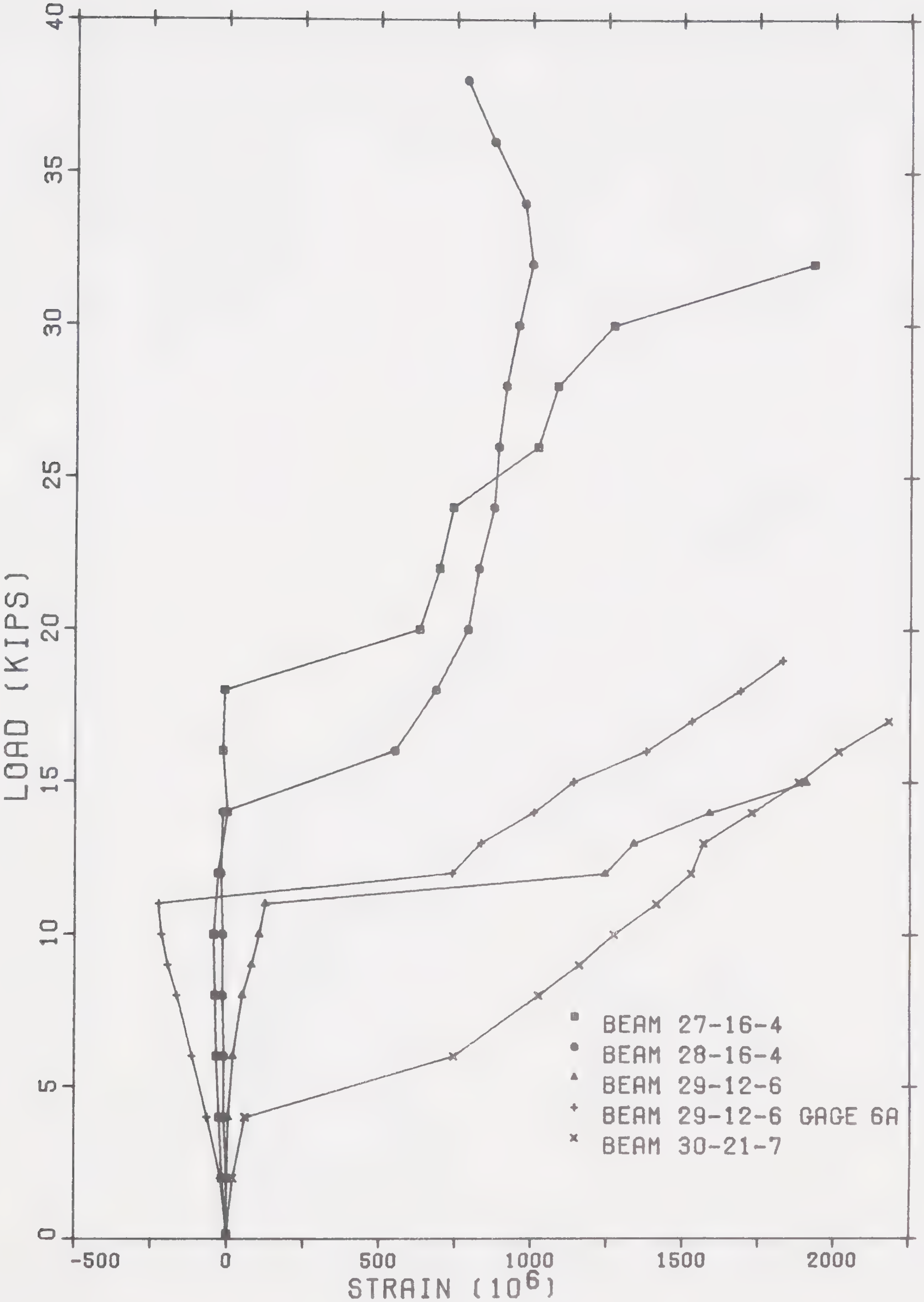


FIG. 4.38 LOAD VS. STRAIN GAGE 6

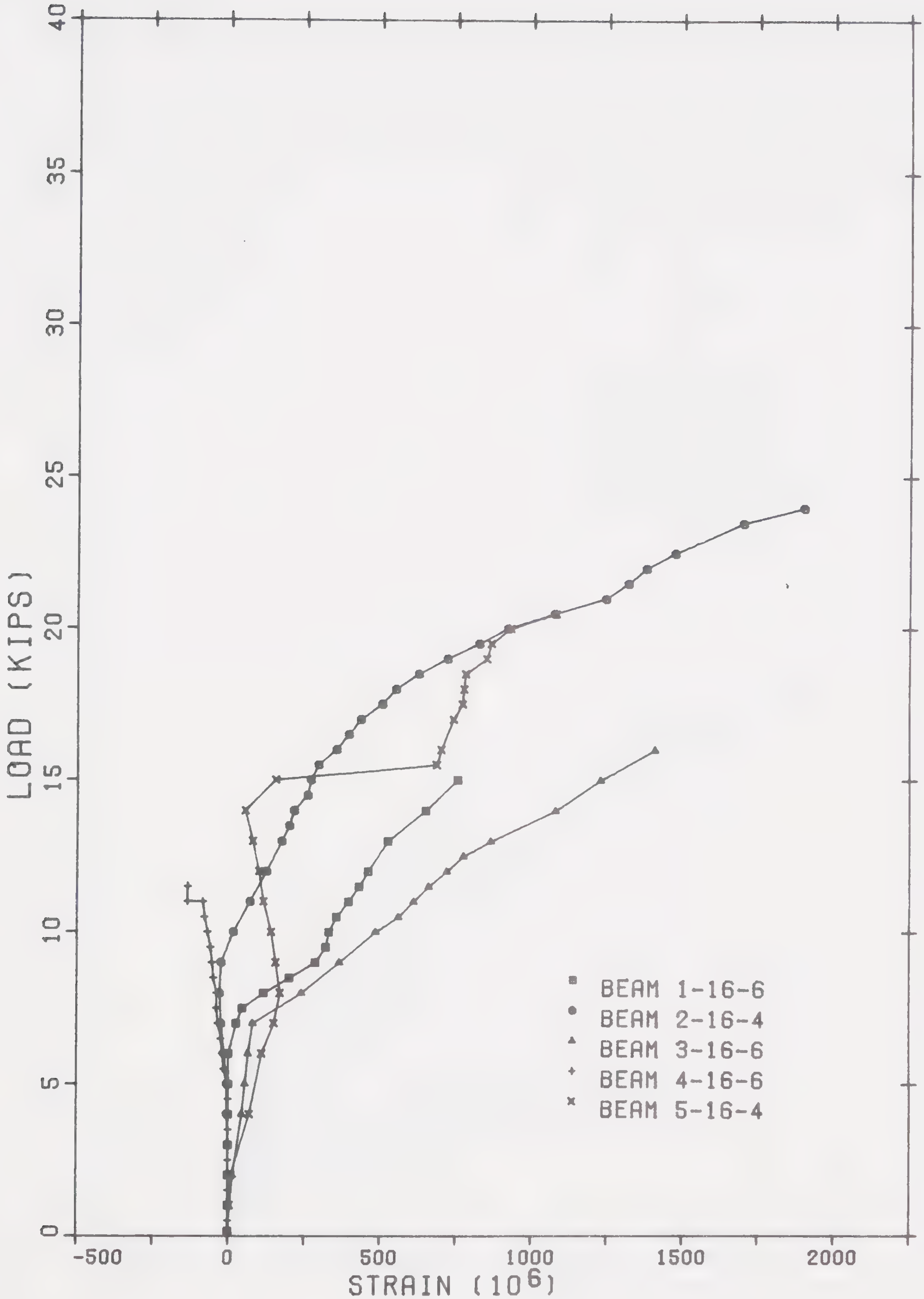


FIG. 4.39 LOAD VS. STRAIN GAGE 7

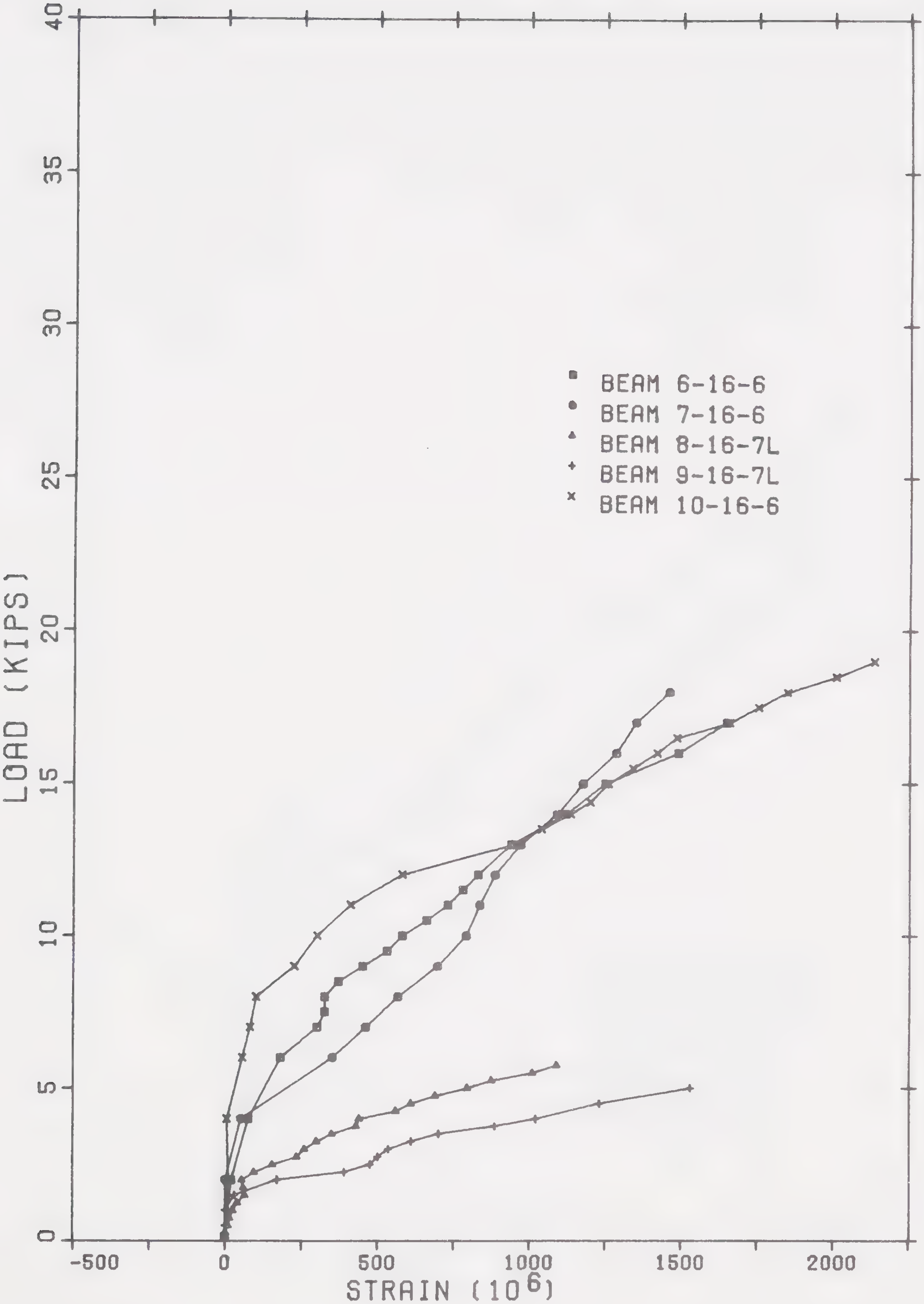


FIG. 4.40 LOAD VS. STRAIN GAGE 7

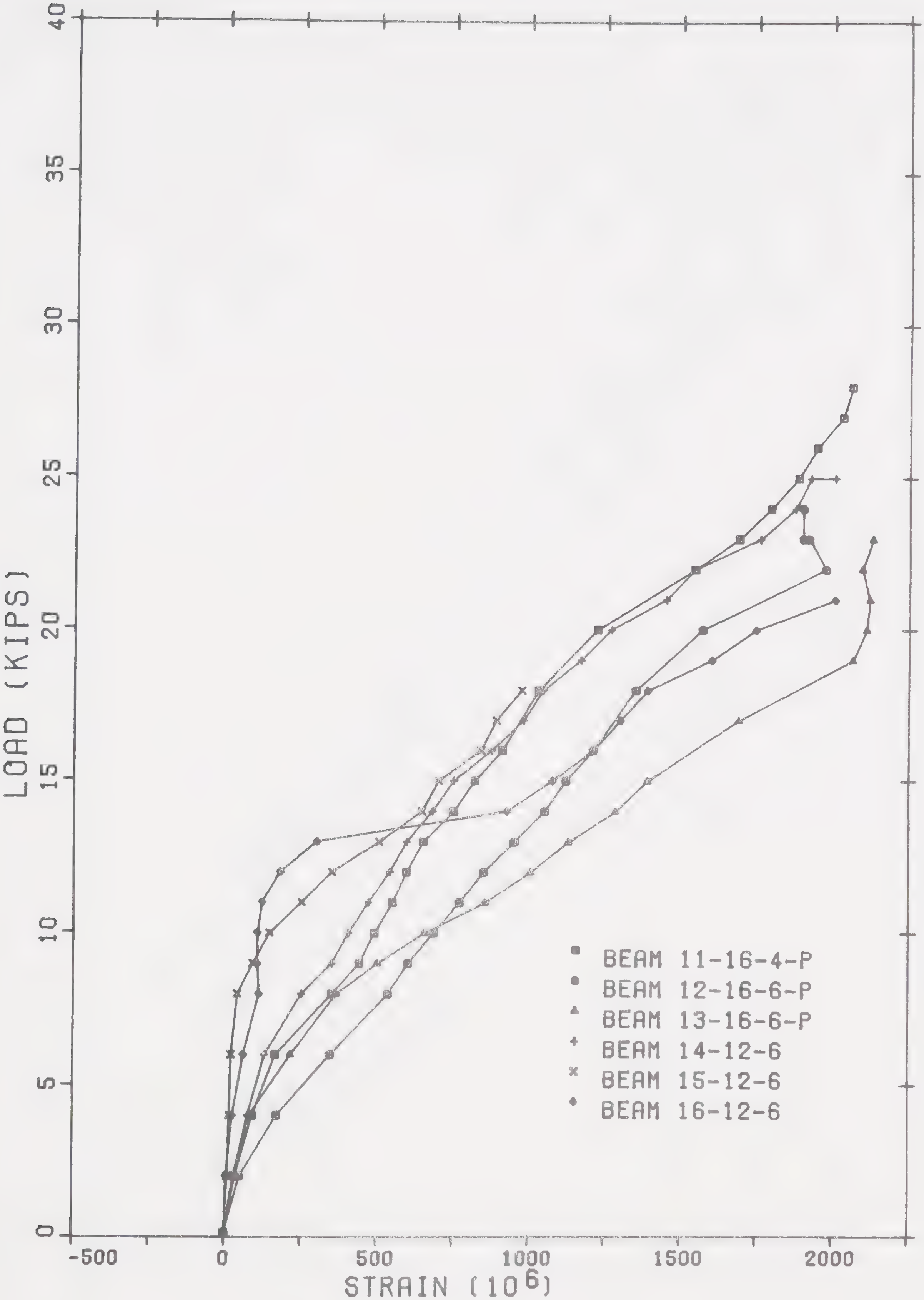


FIG. 4.41 LOAD VS. STRAIN GAGE 7

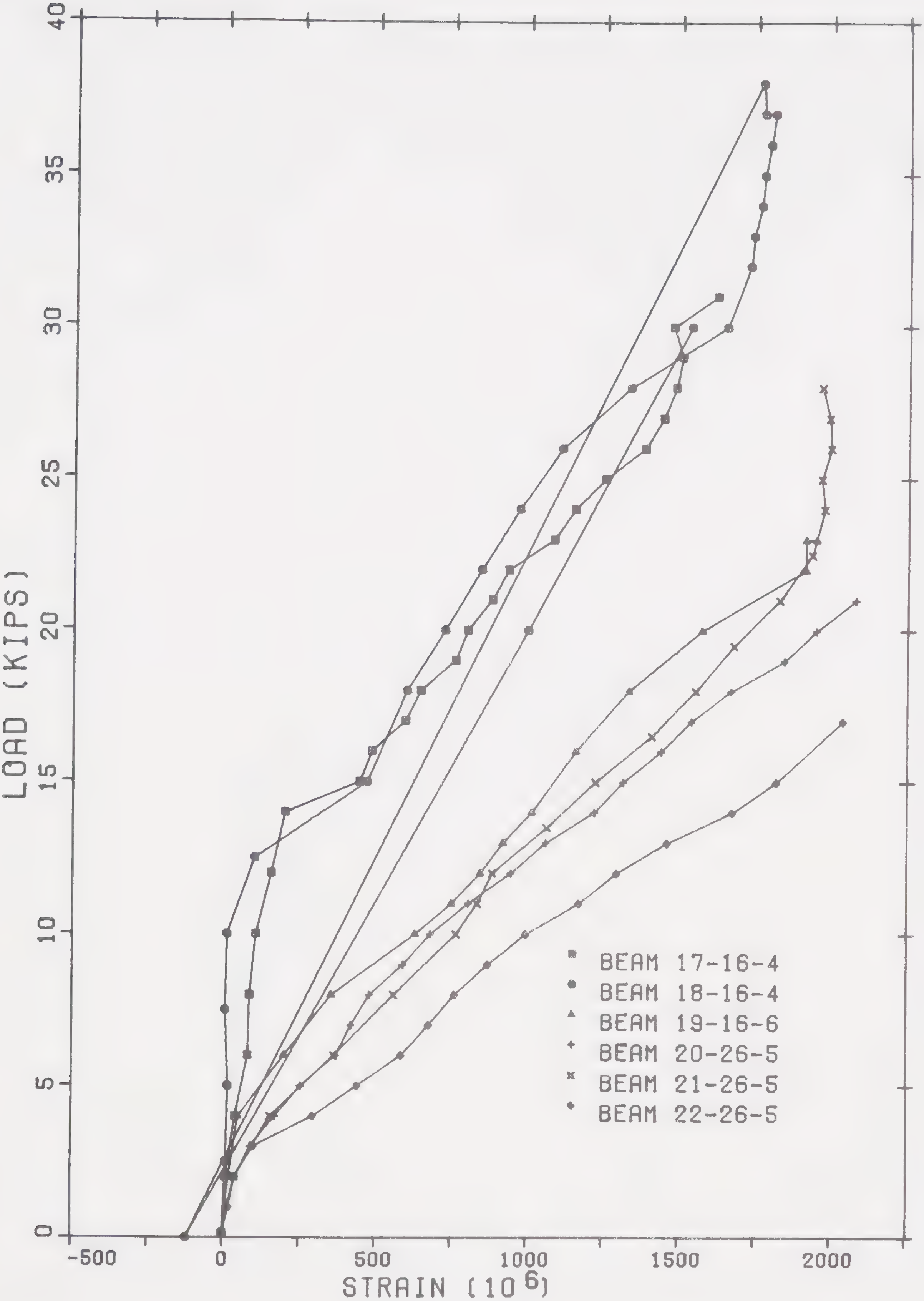


FIG. 4.42 LOAD VS. STRAIN GAGE 7

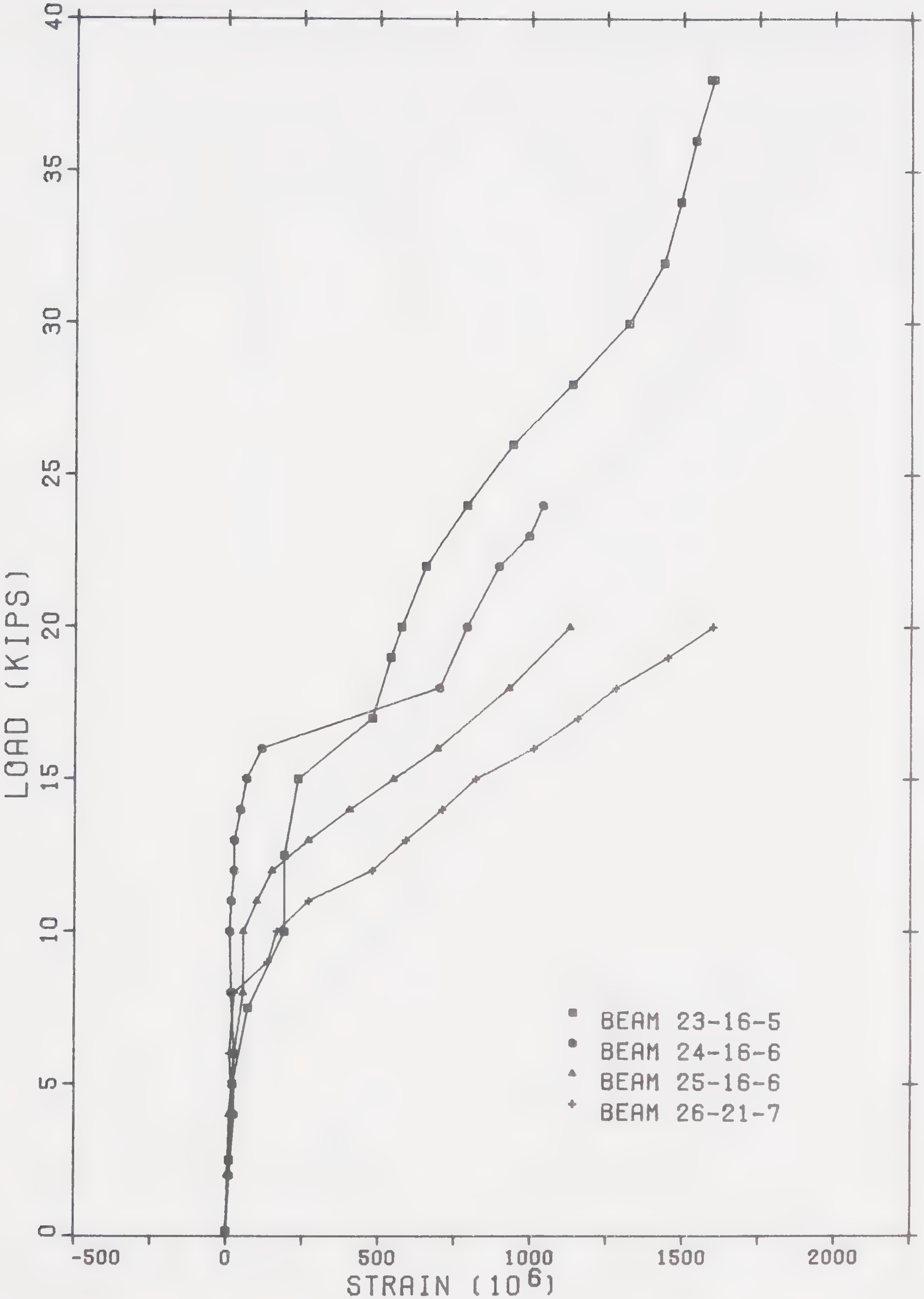


FIG. 4.43 LOAD VS. STRAIN GAGE 7

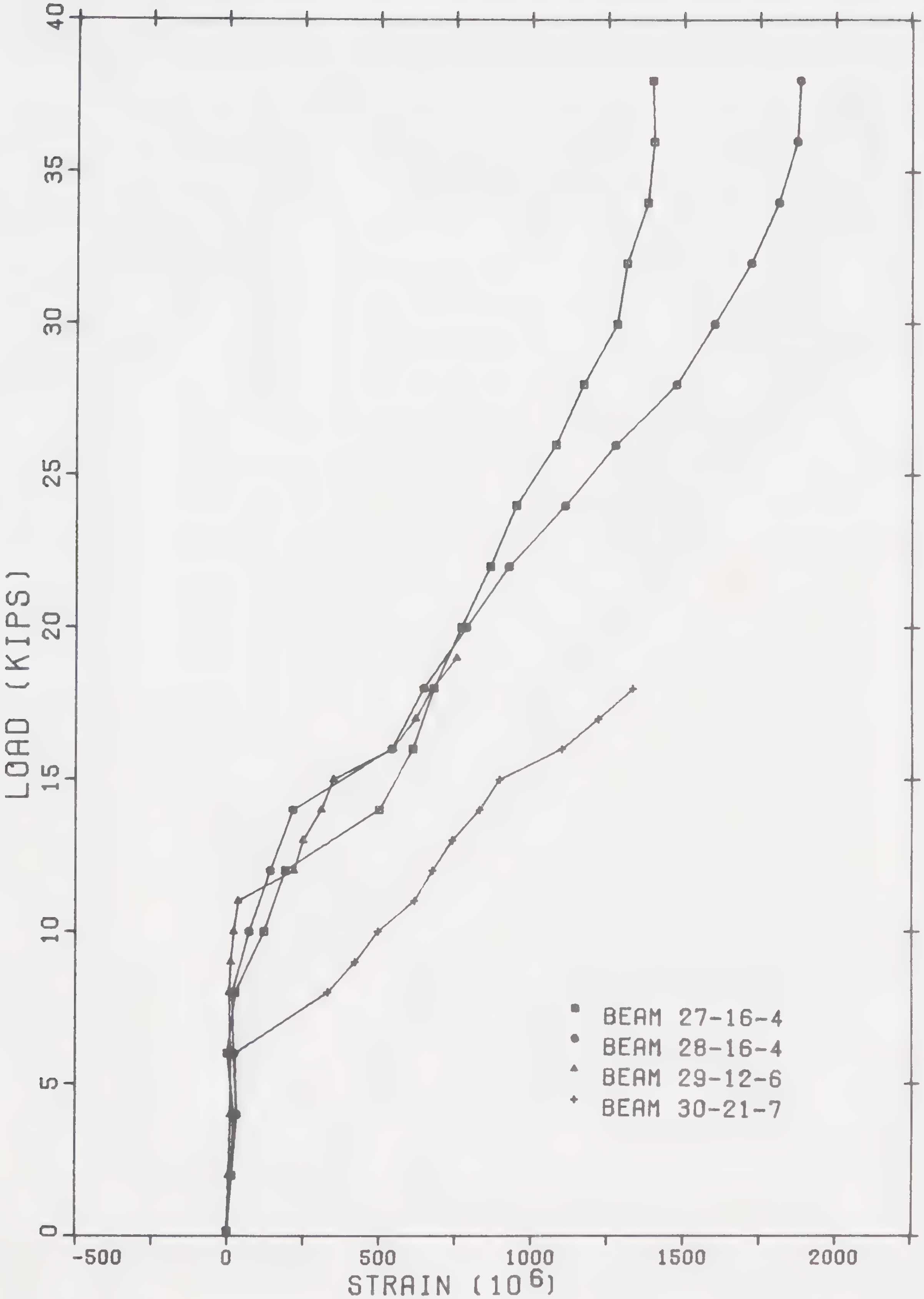


FIG. 4.44 LOAD VS. STRAIN GAGE 7

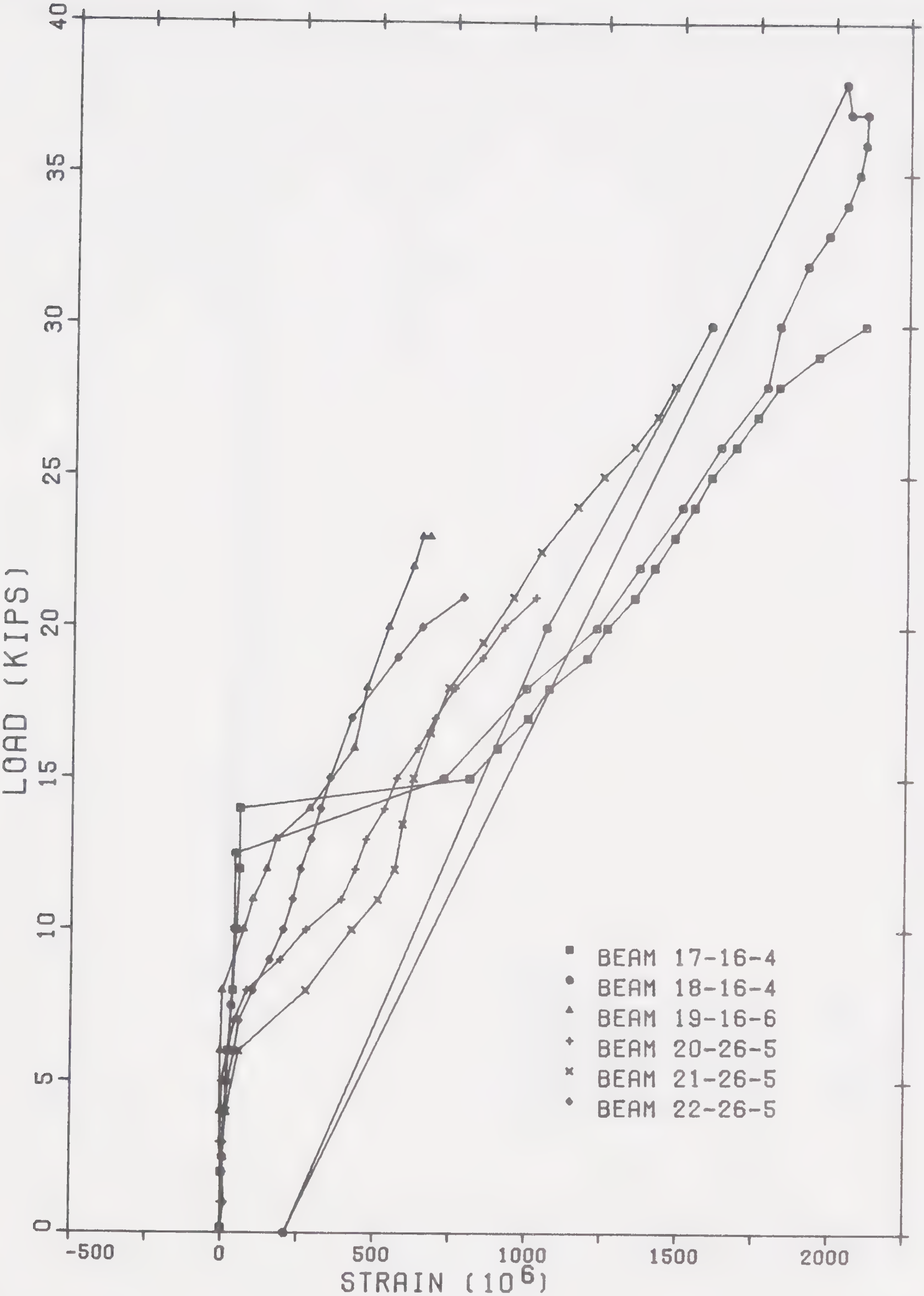


FIG. 4.45 LOAD VS. STRAIN GAGE 8

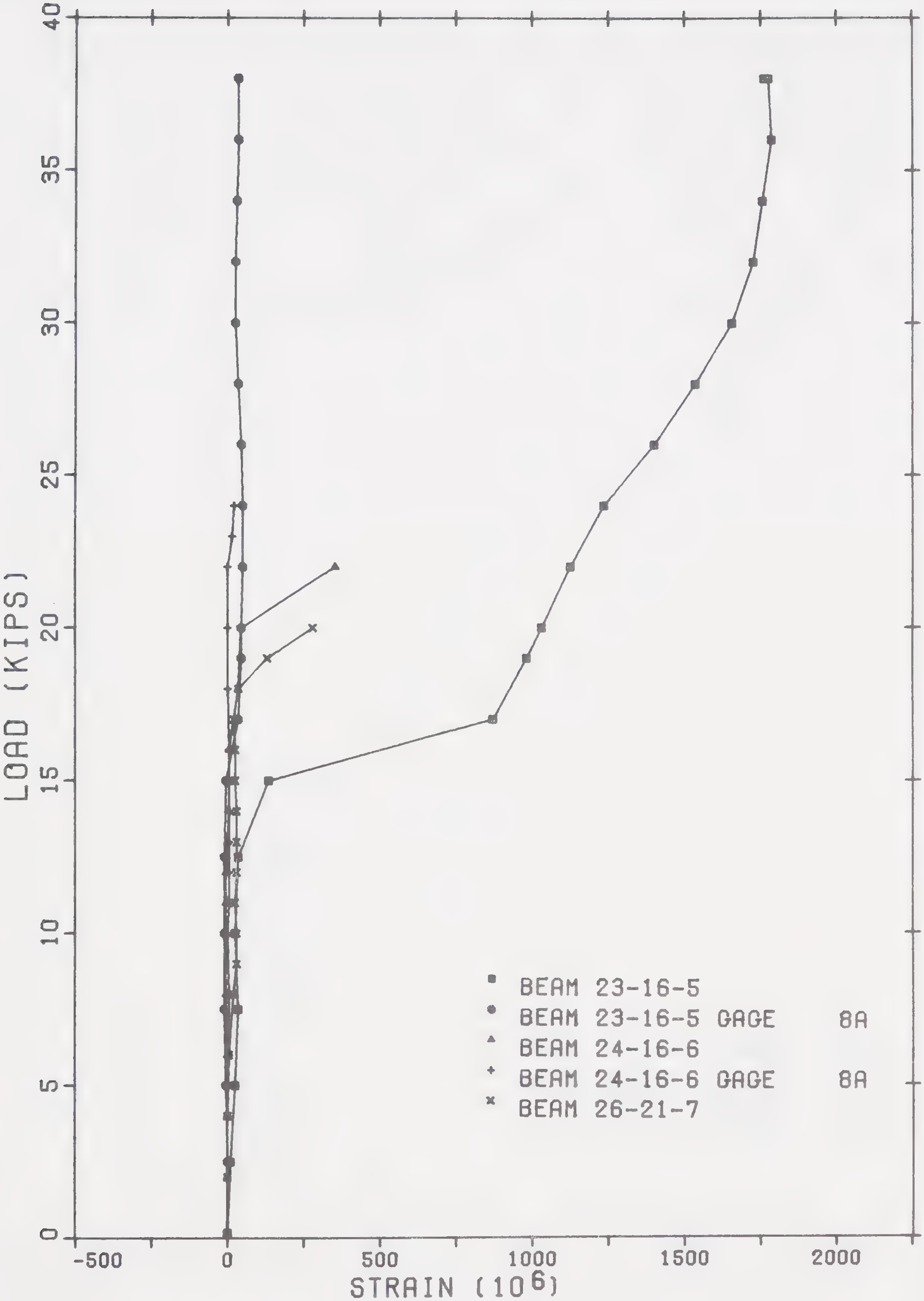


FIG. 4.46 LOAD VS. STRAIN GAGE 8

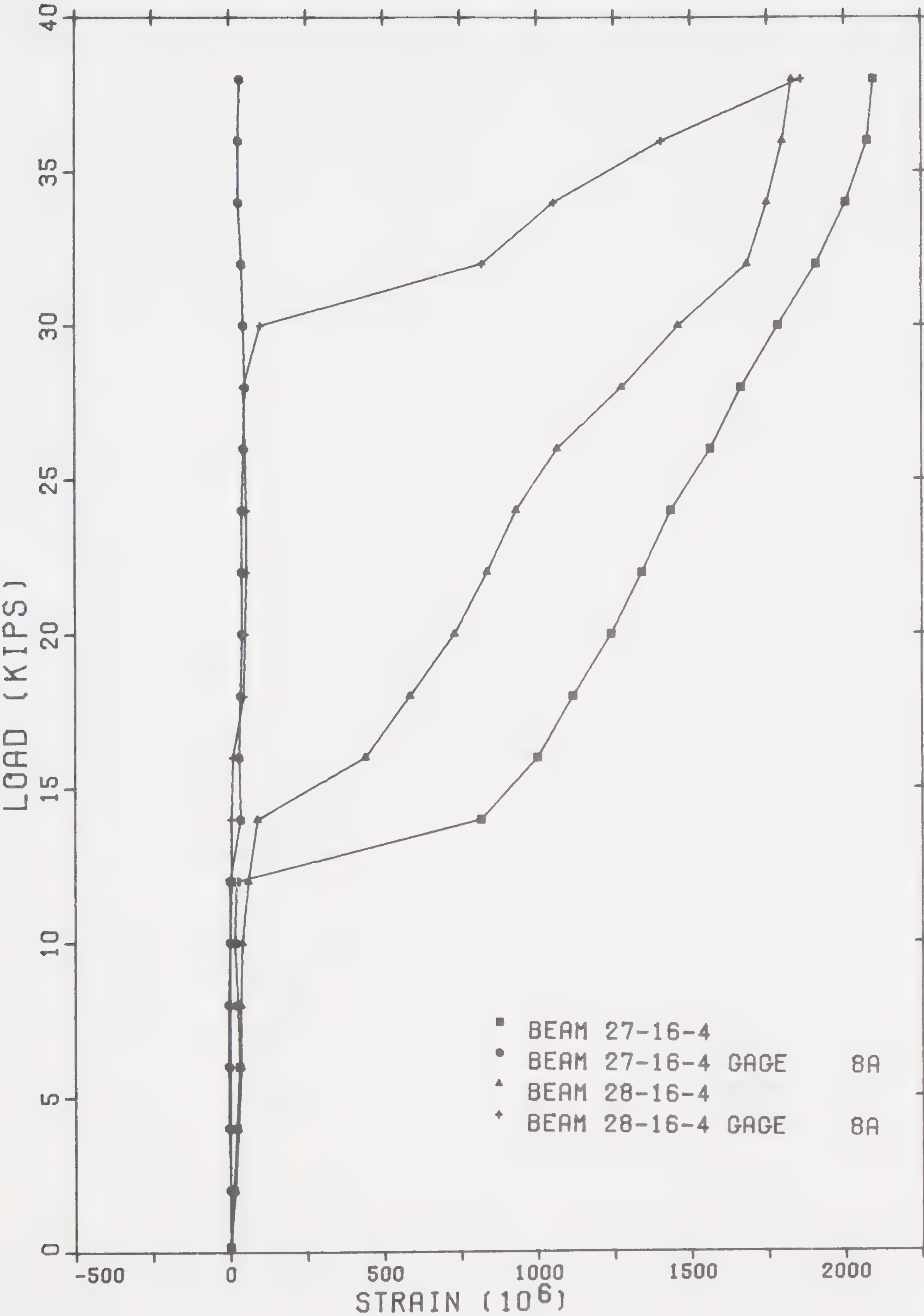


FIG. 4.47 LOAD VS. STRAIN GAGE 8

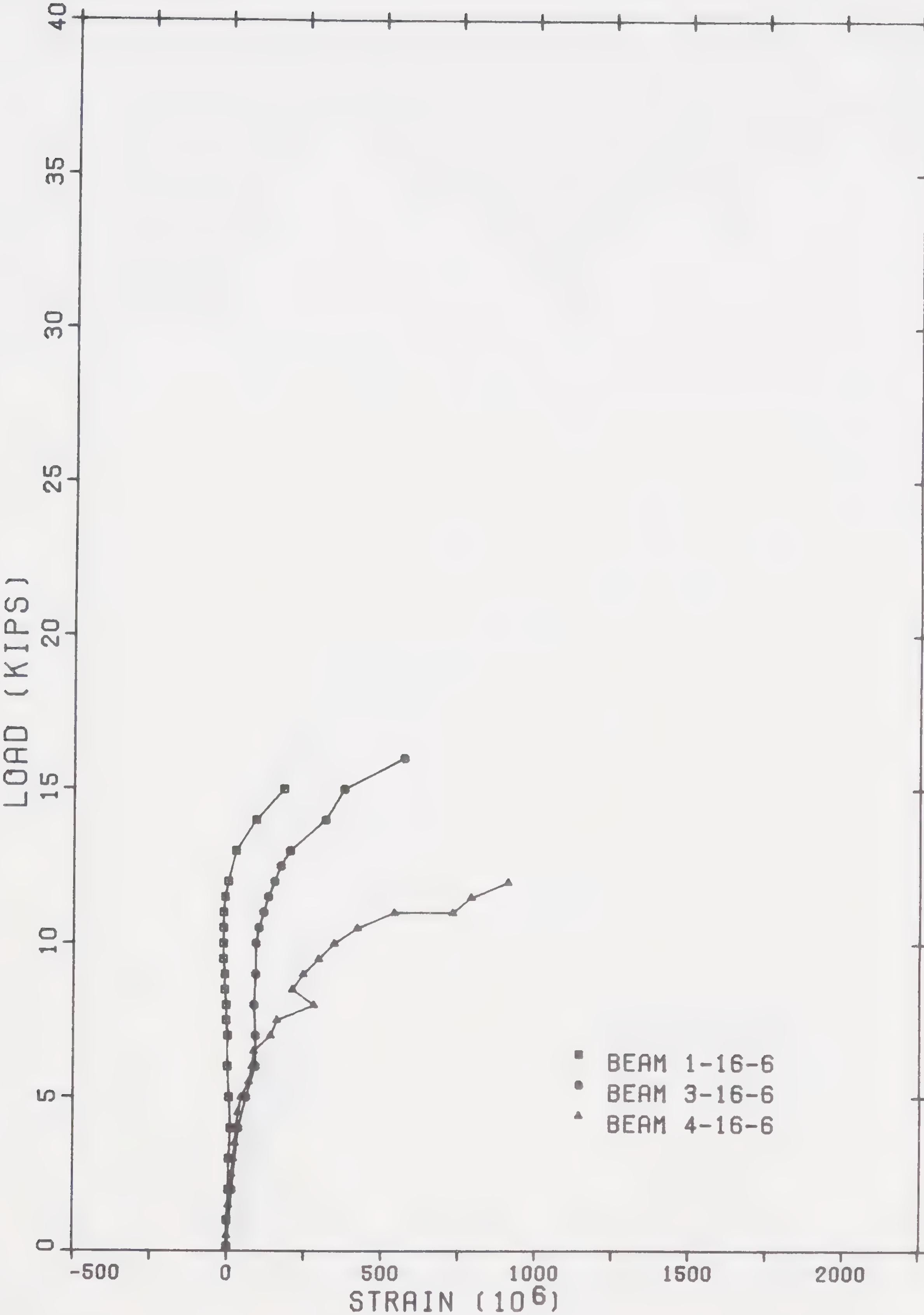


FIG. 4.48 LOAD VS. STRAIN GAGE 9

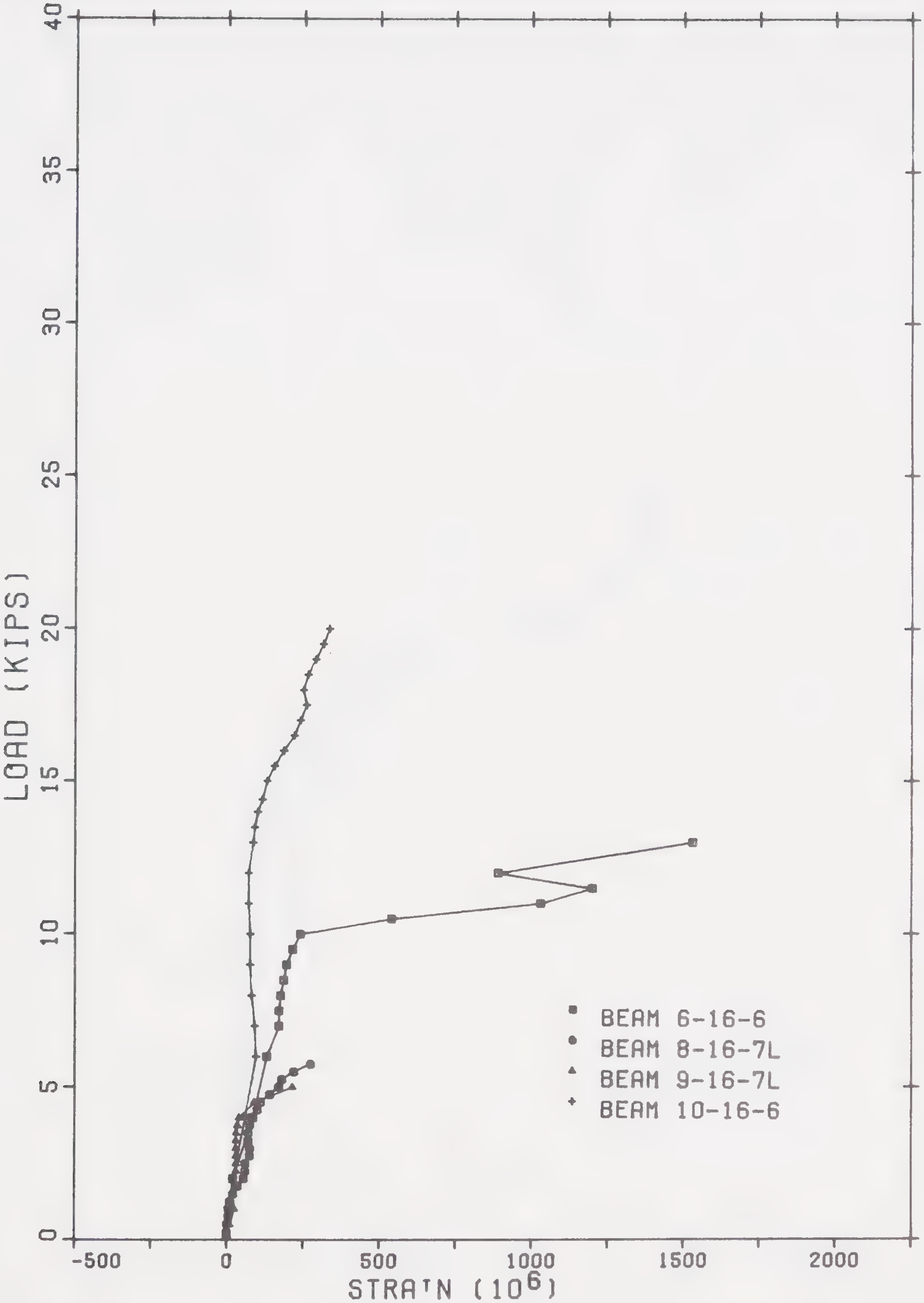


FIG. 4.49 LOAD VS. STRAIN GAGE 9

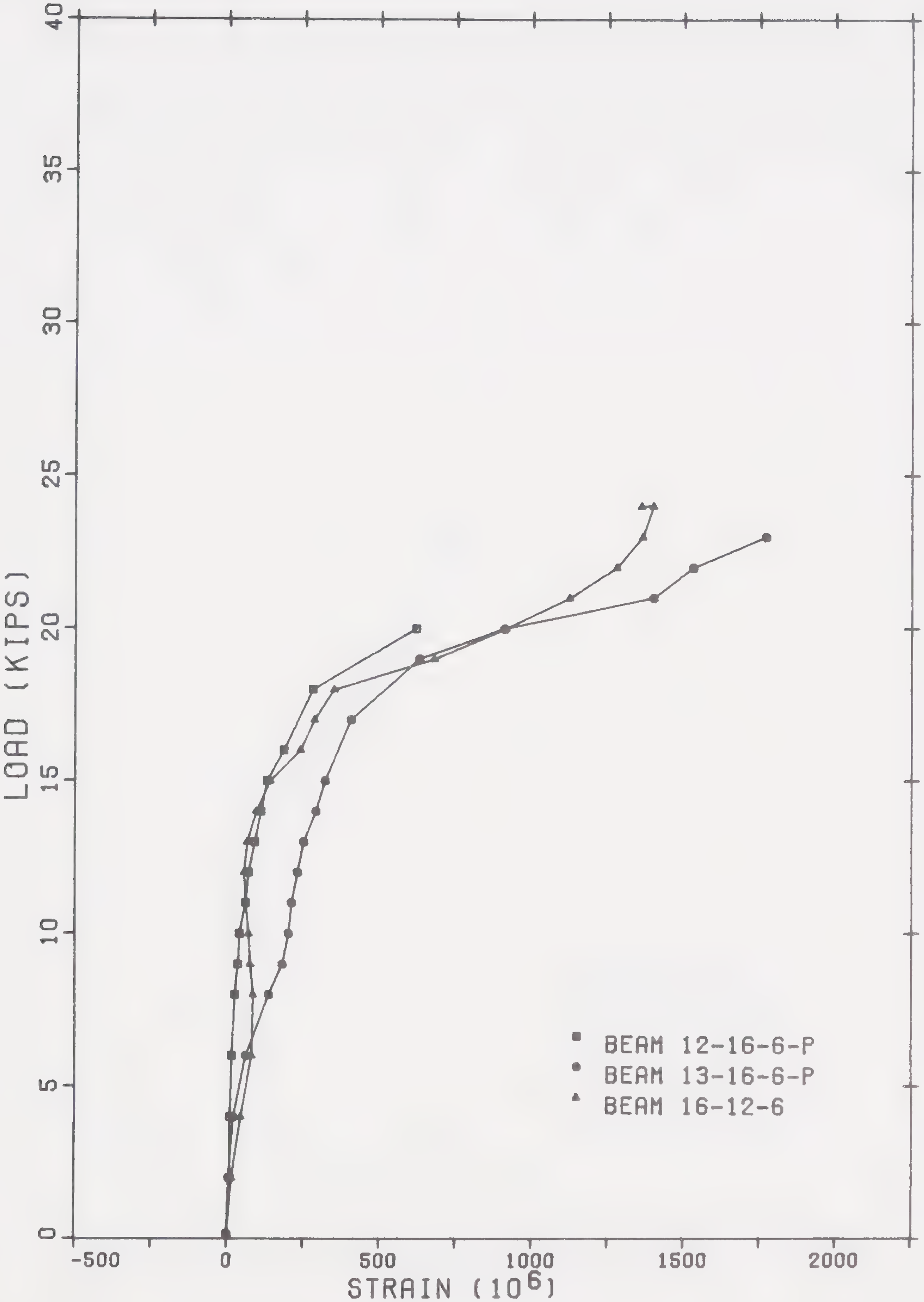


FIG. 4.50 LOAD VS. STRAIN GAGE 9

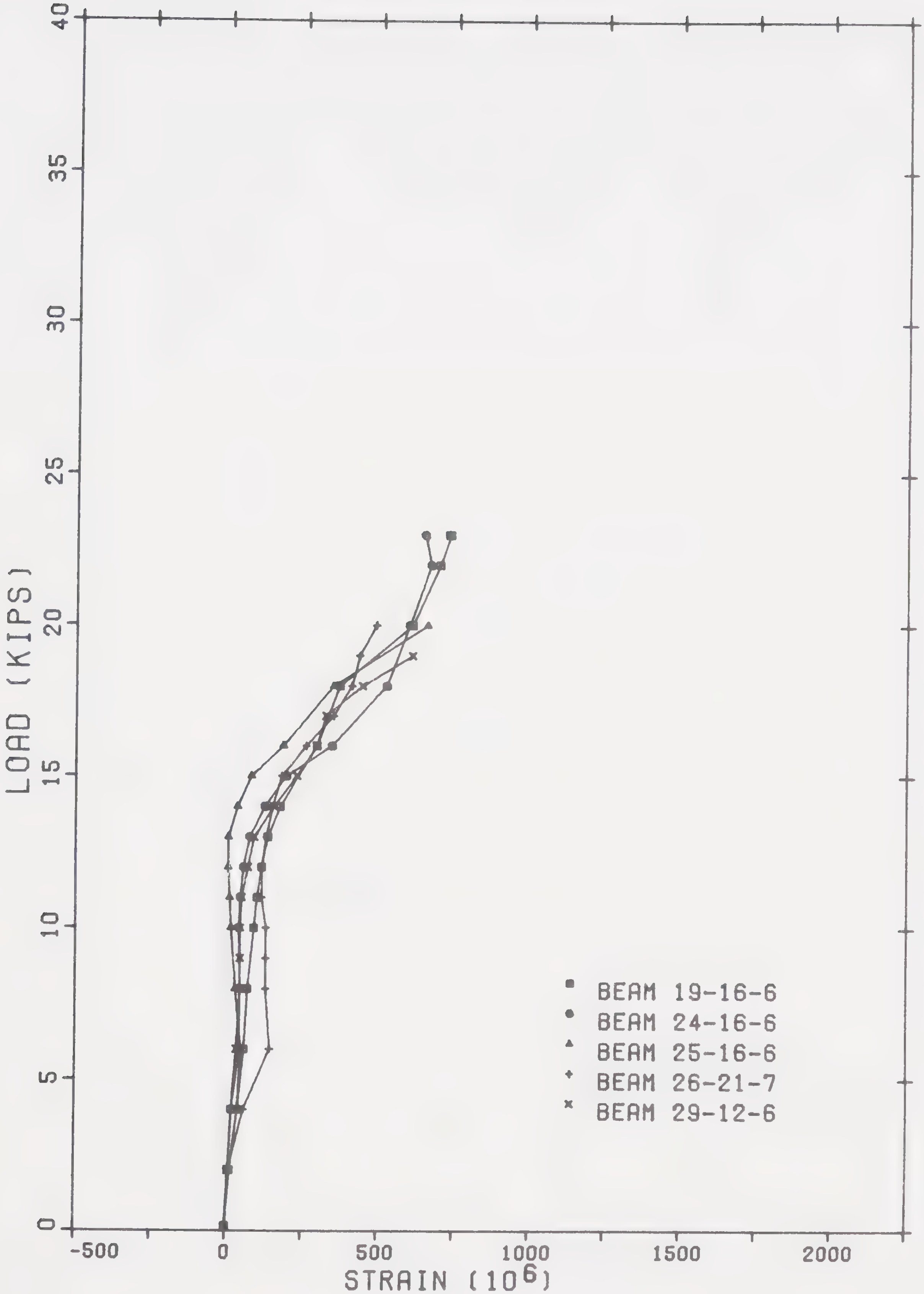


FIG. 4.51 LOAD VS. STRAIN GAGE 9

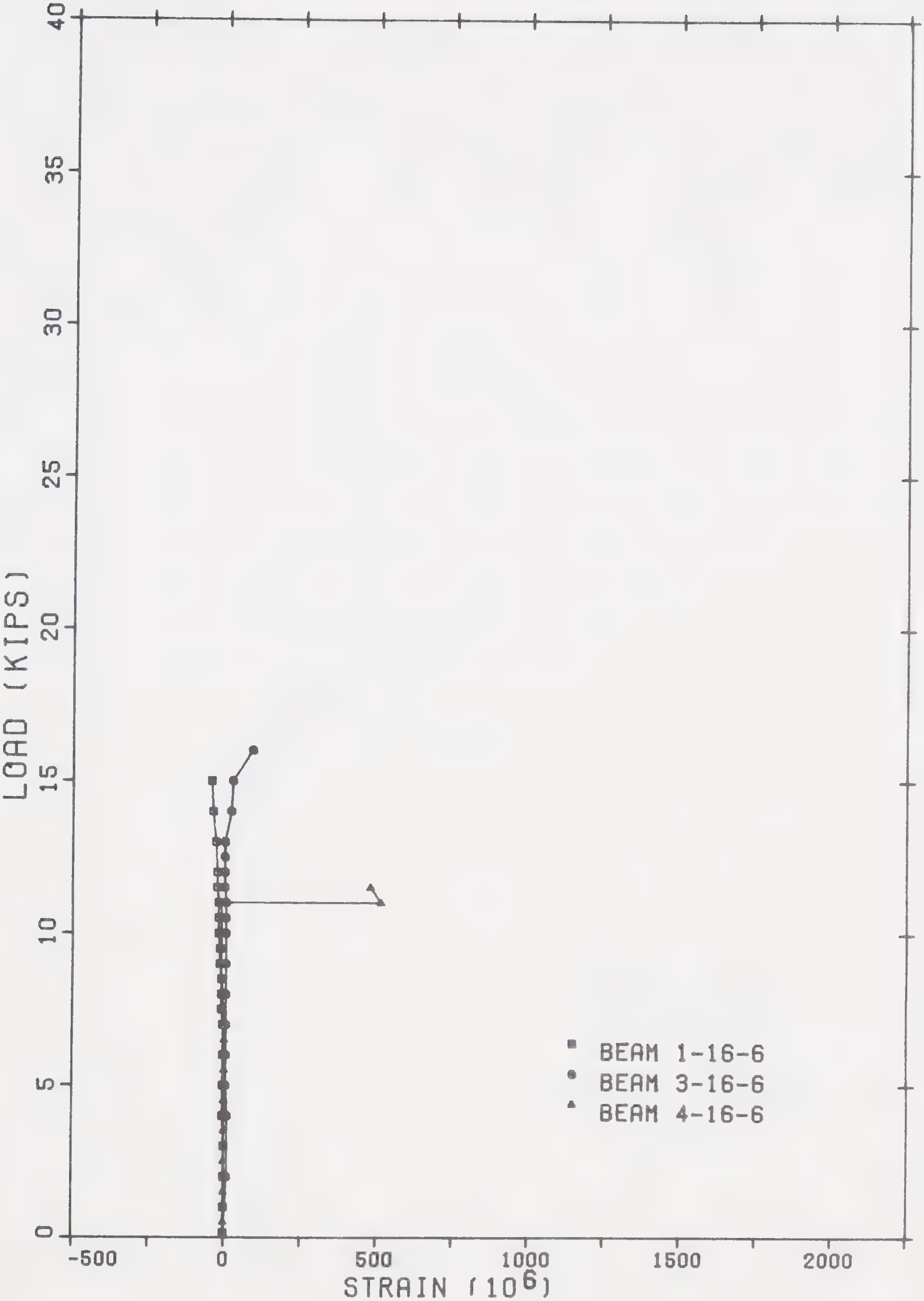


FIG. 4.52 LOAD VS. STRAIN GAGE 10

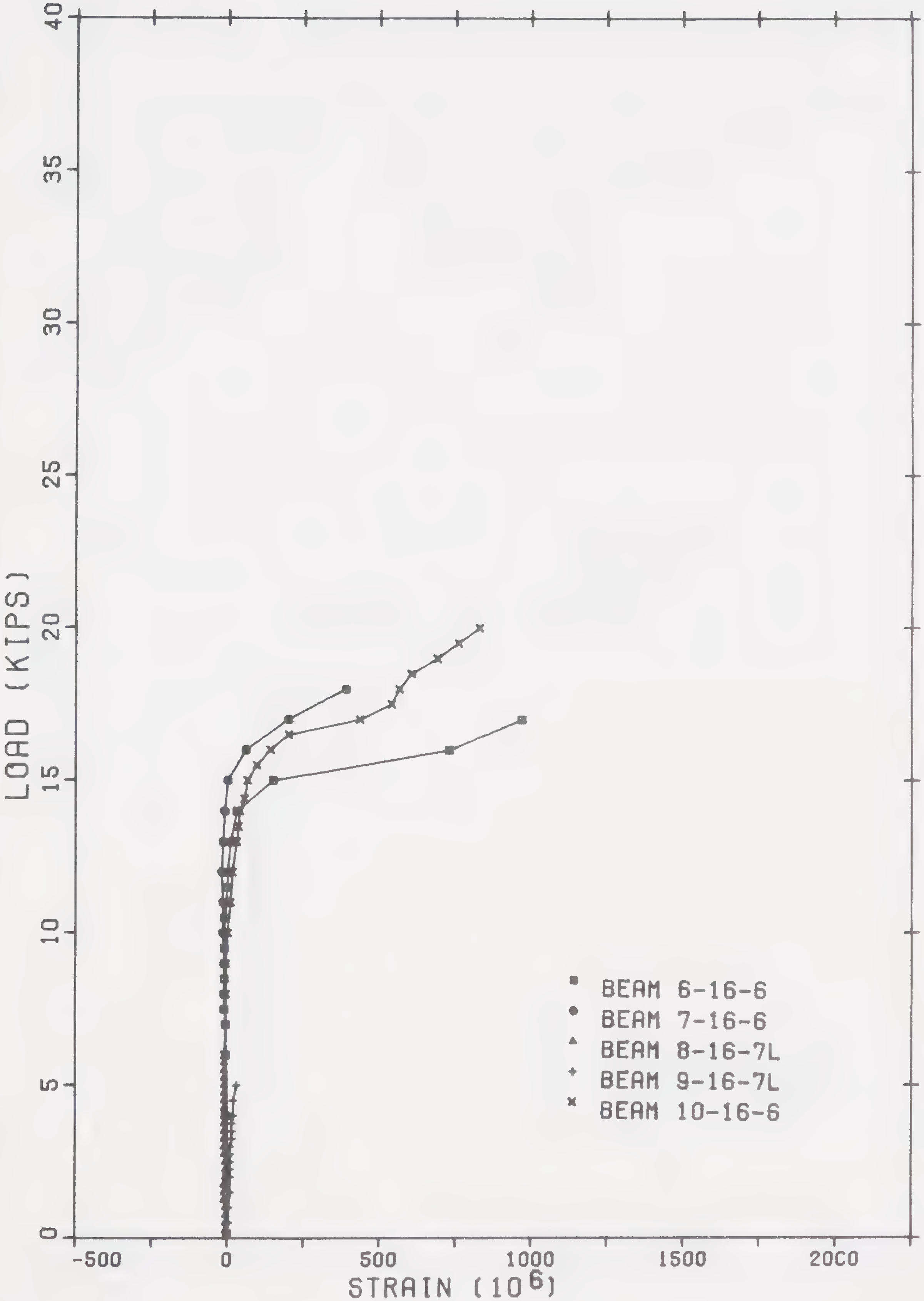


FIG. 4.53 LOAD VS. STRAIN GAGE 10

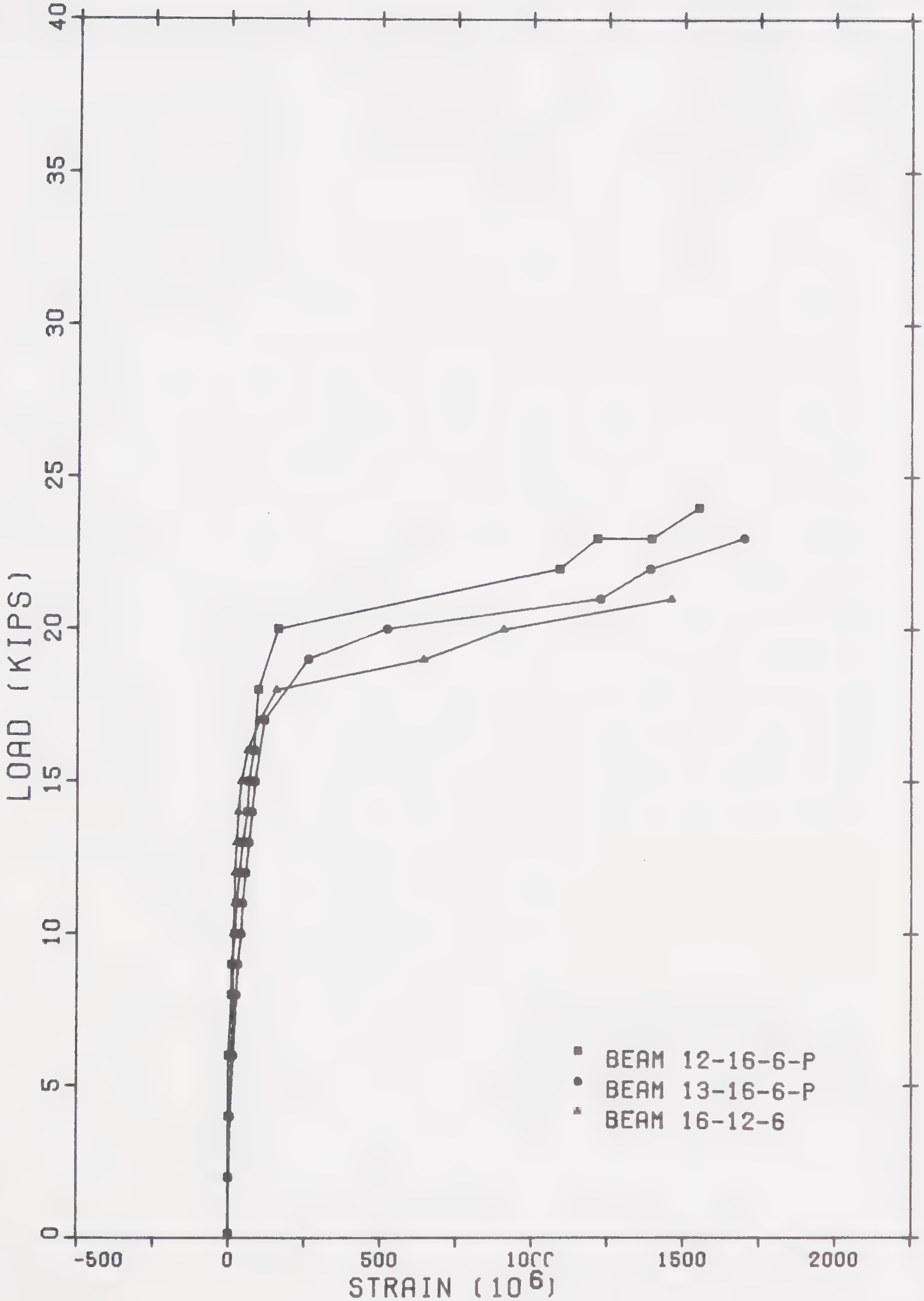


FIG. 4.54 LOAD VS. STRAIN GAGE 10

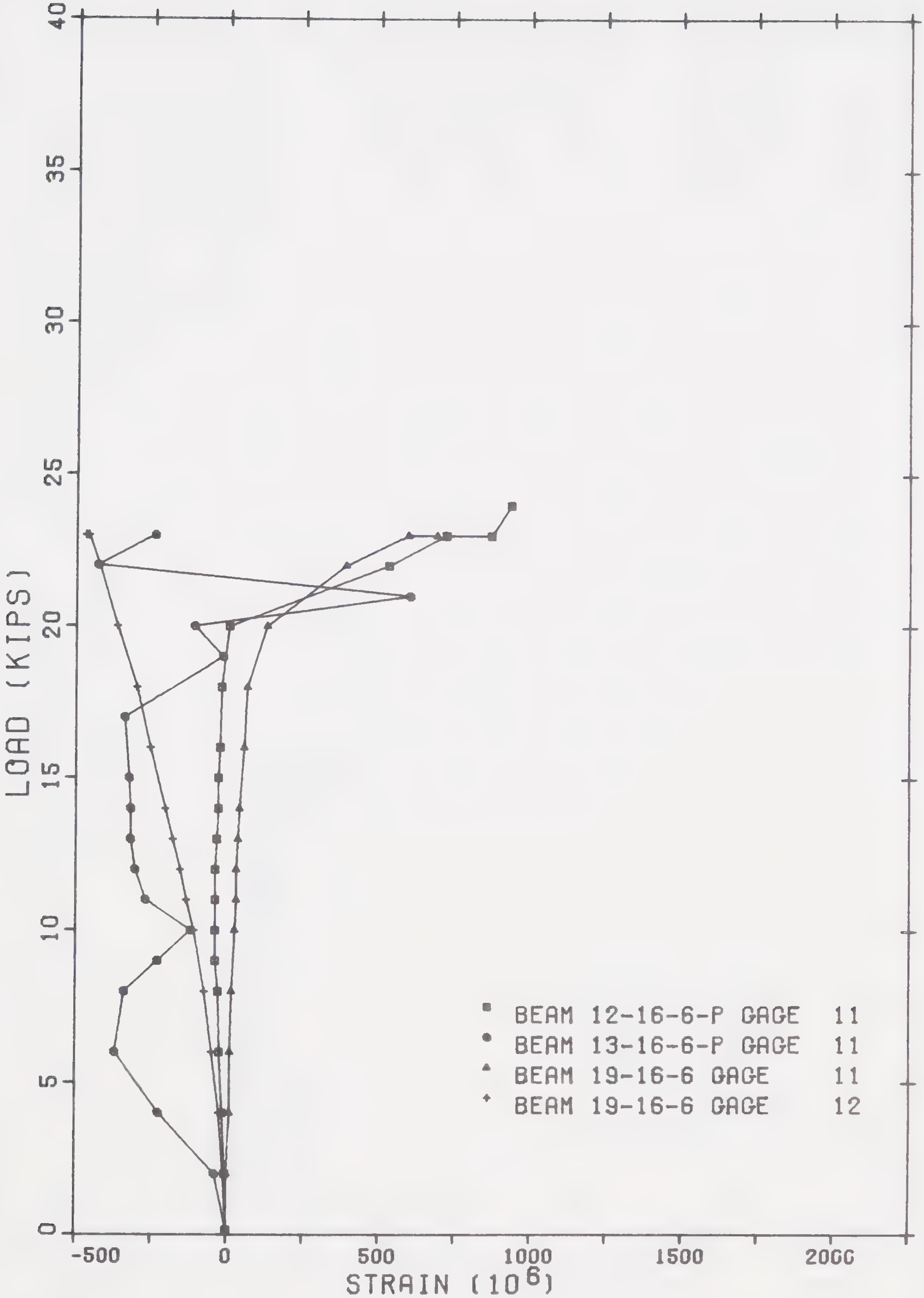


FIG. 4.55 LOAD VS. STRAIN GAGES 11 AND 12

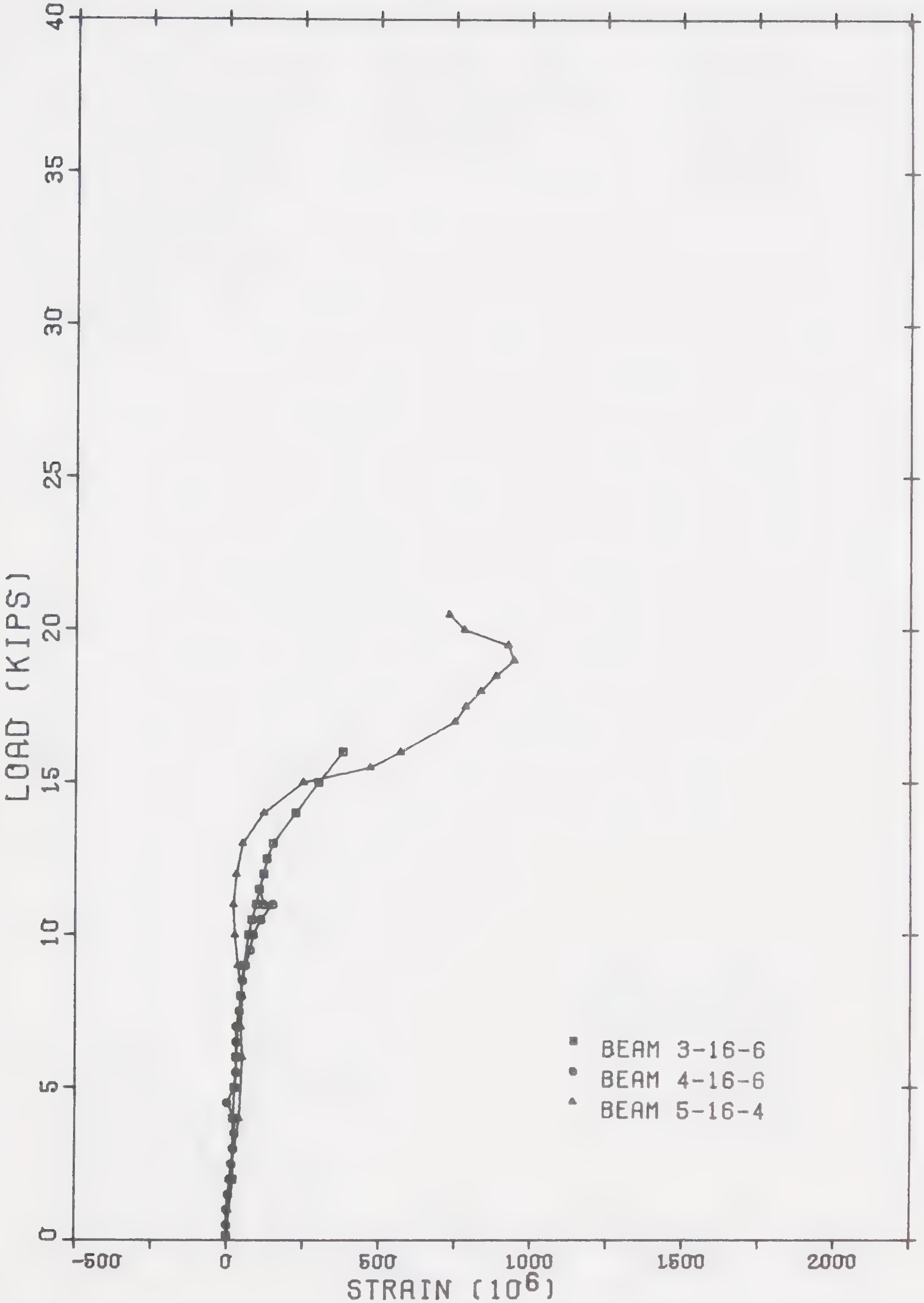


FIG. 4.56 LOAD VS. STRAIN GAGE 13

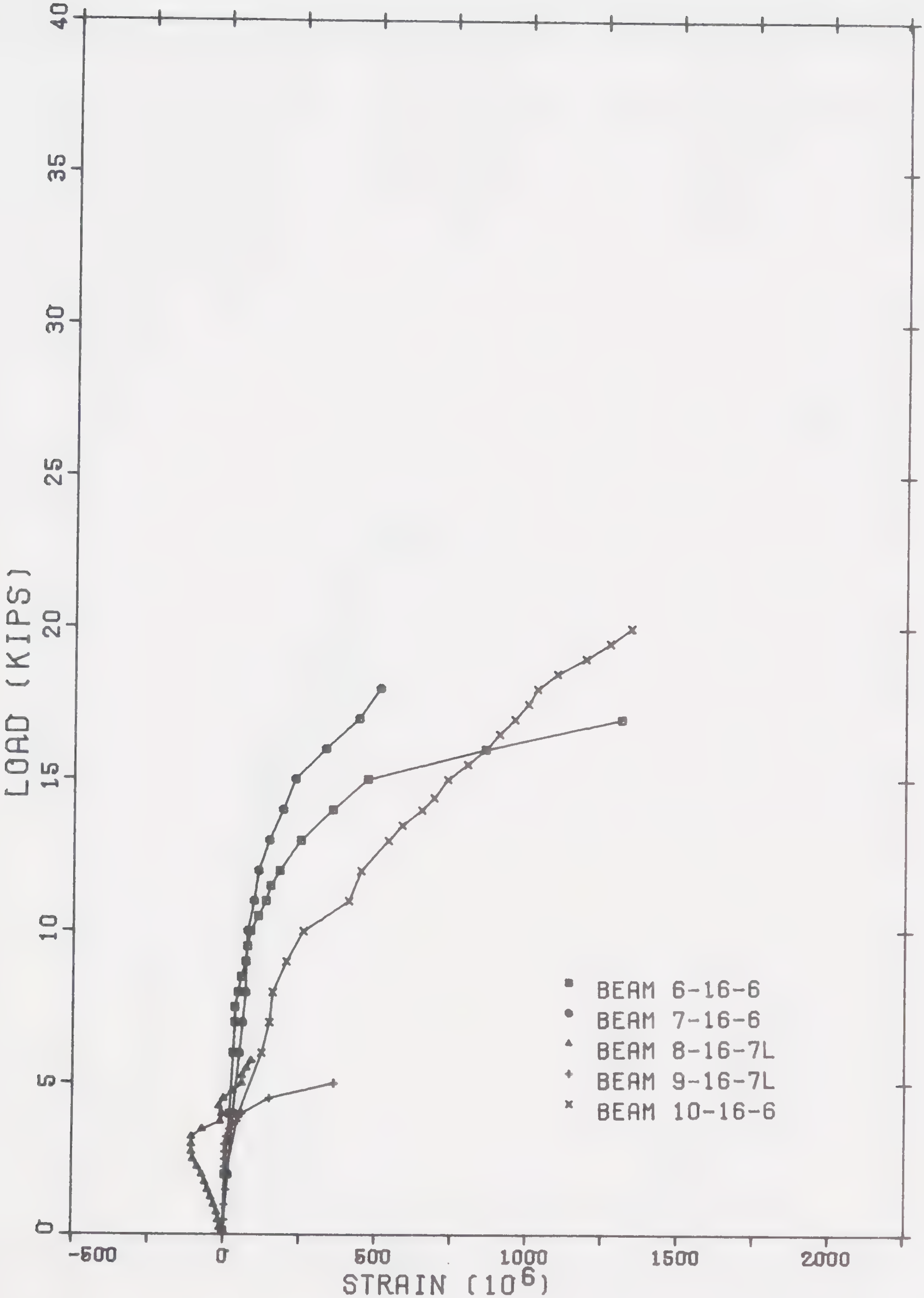


FIG. 4.57 LOAD VS. STRAIN GAGE 13

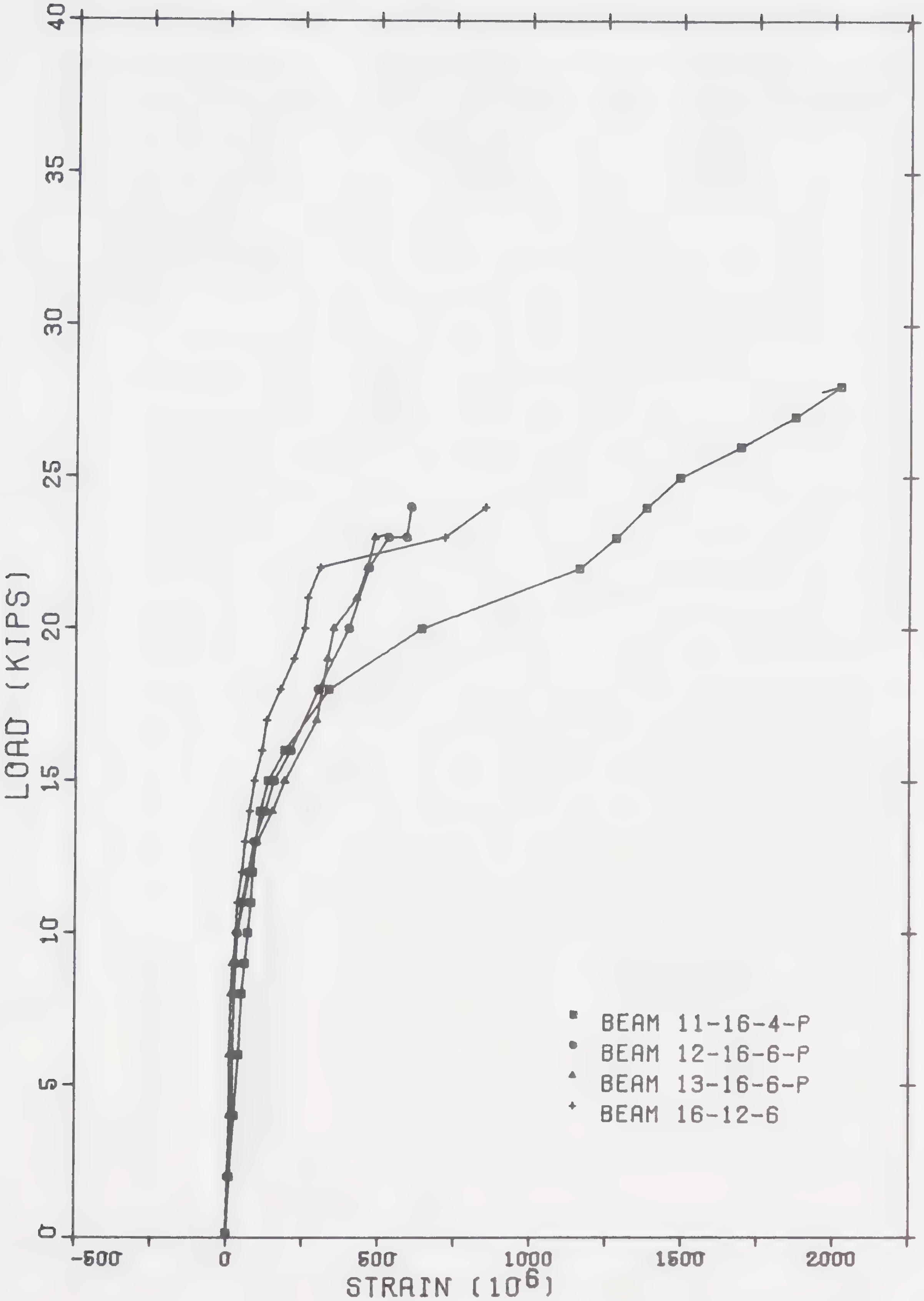


FIG. 4.58 LOAD VS. STRAIN GAGE 13

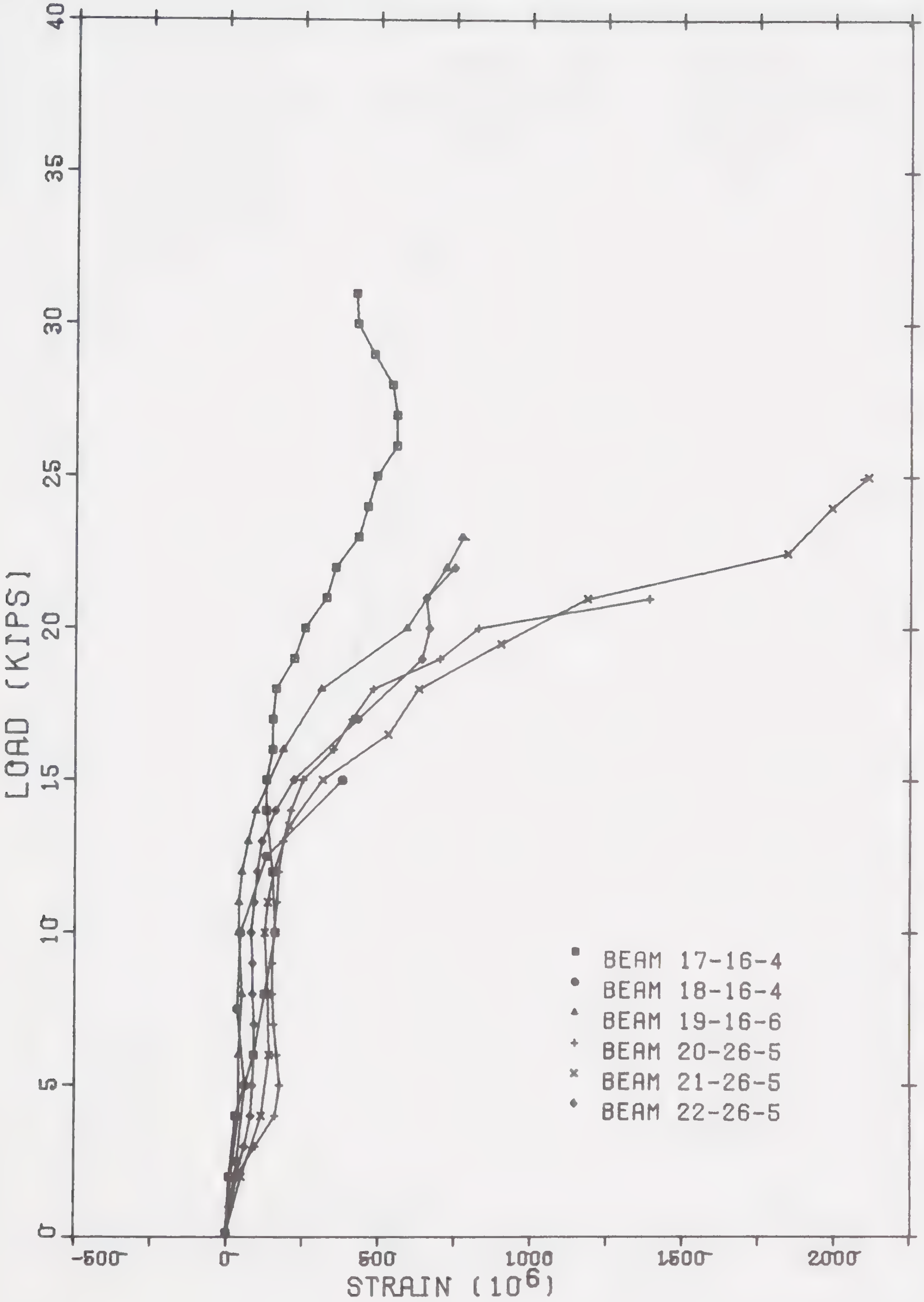


FIG. 4.59 LOAD VS. STRAIN GAGE 13

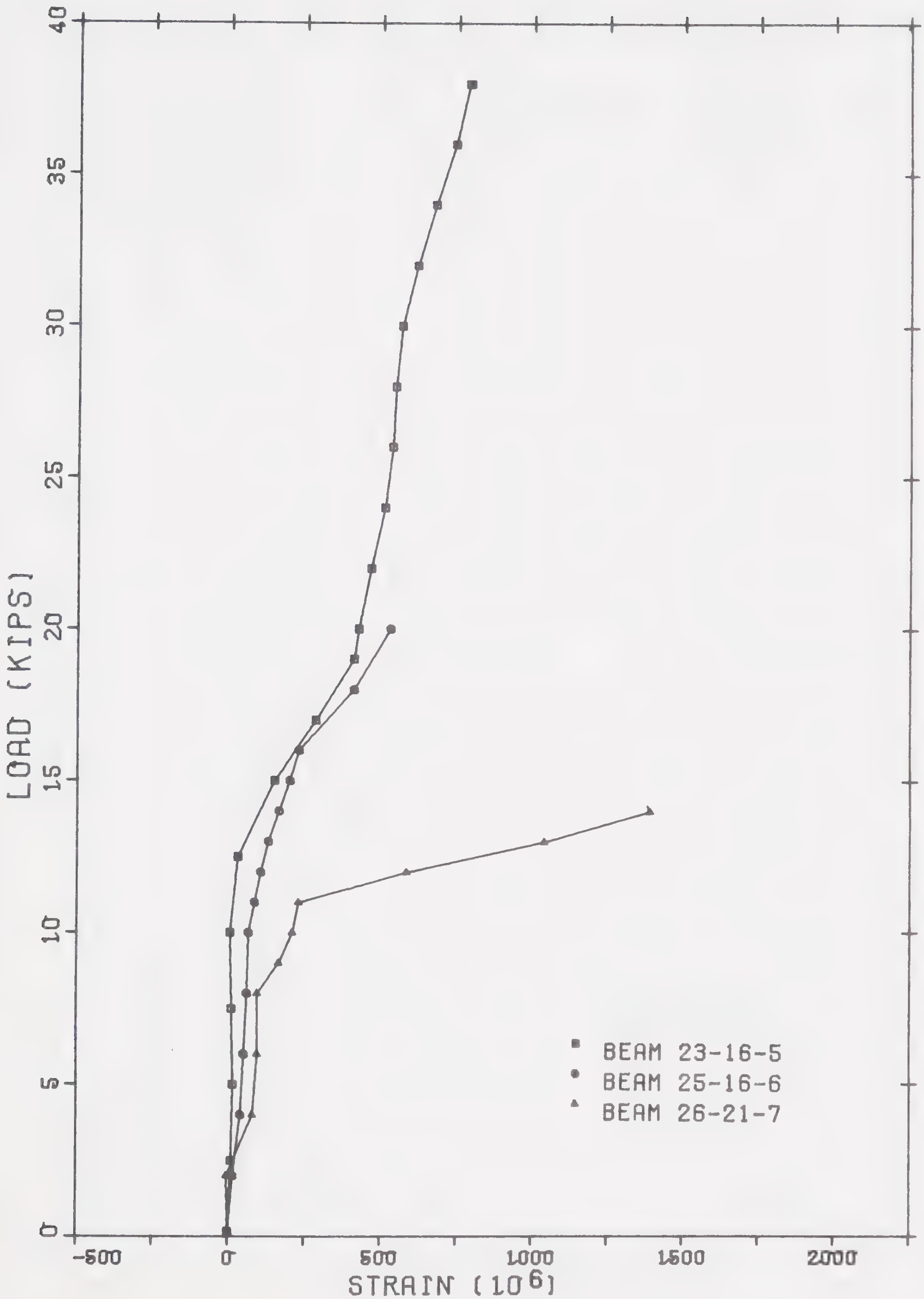


FIG. 4.60 LOAD VS. STRAIN GAGE 13

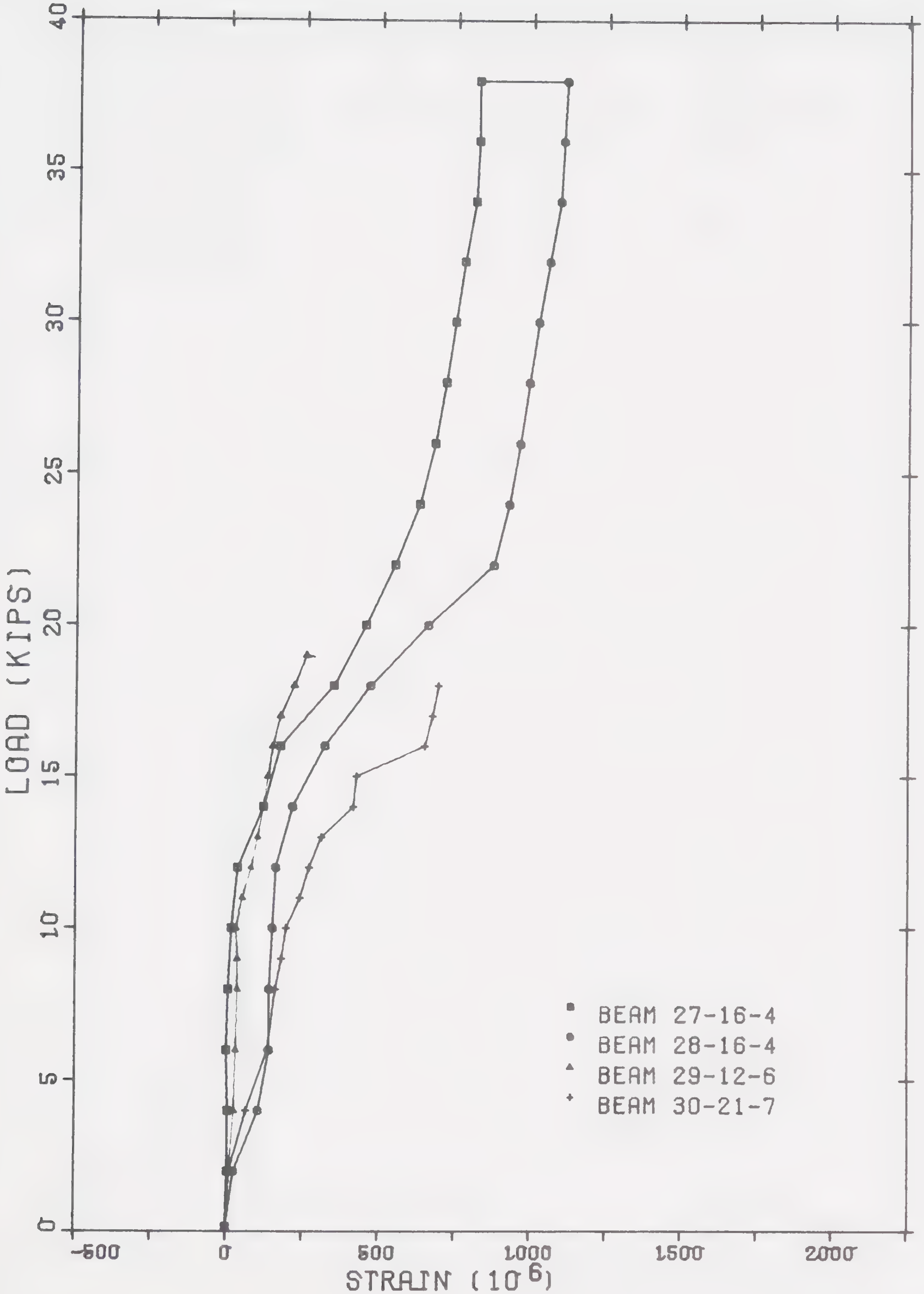


FIG. 4.61 LOAD VS. STRAIN GAGE 13

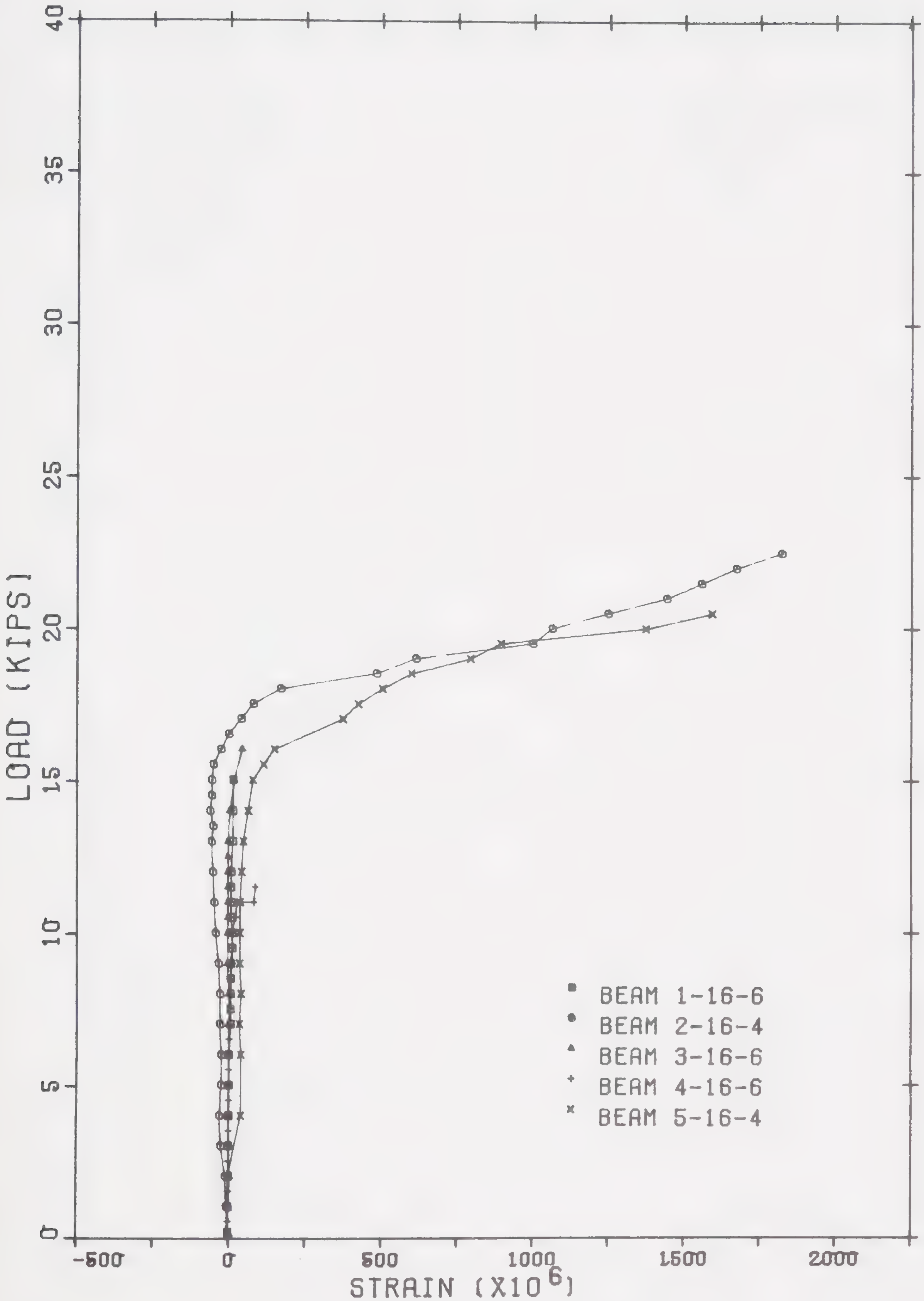


FIG. 4.62 LOAD VS. STRAIN FOR GAGE 14

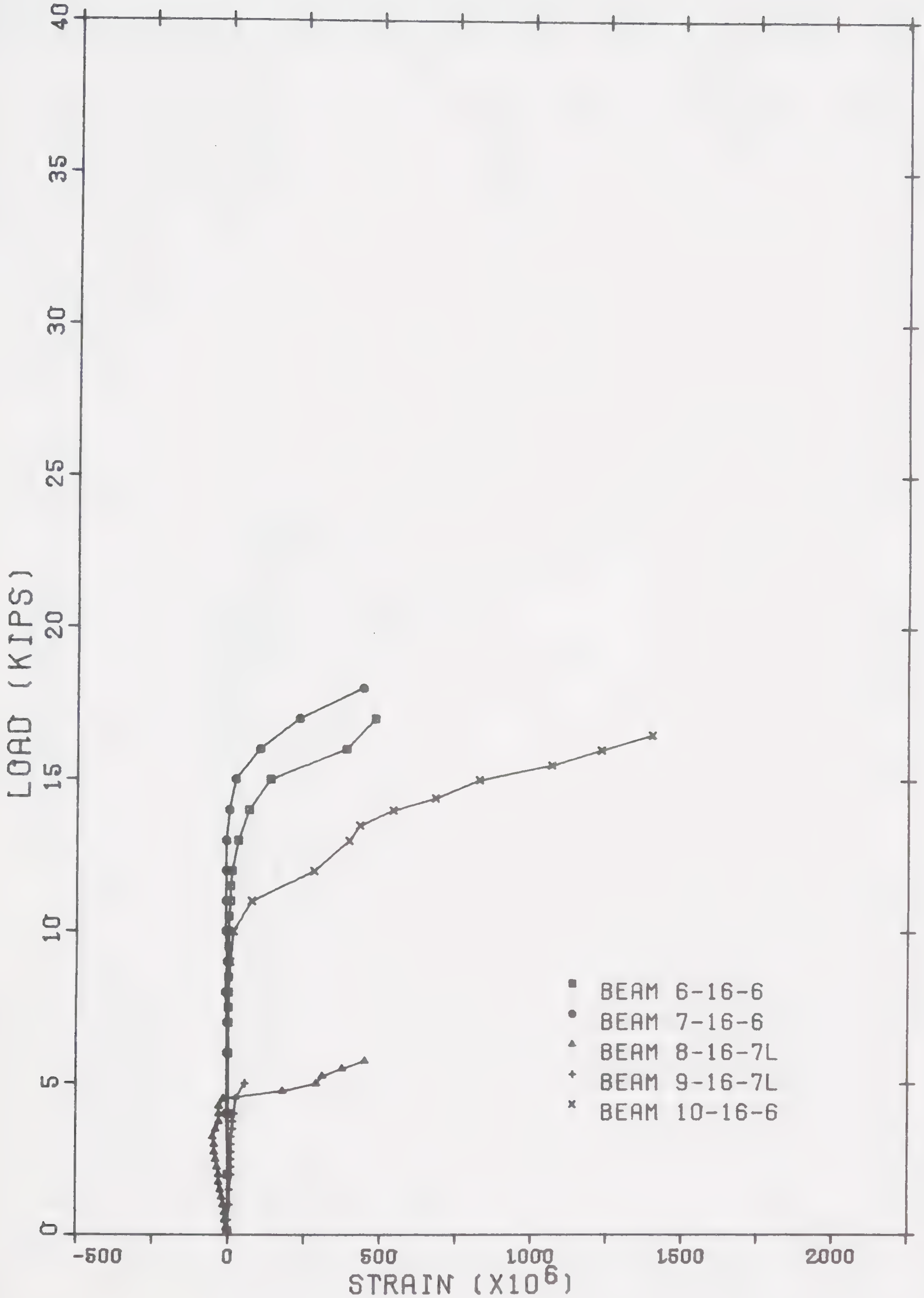


FIG. 4.63 LOAD VS. STRAIN FOR GAGE 14

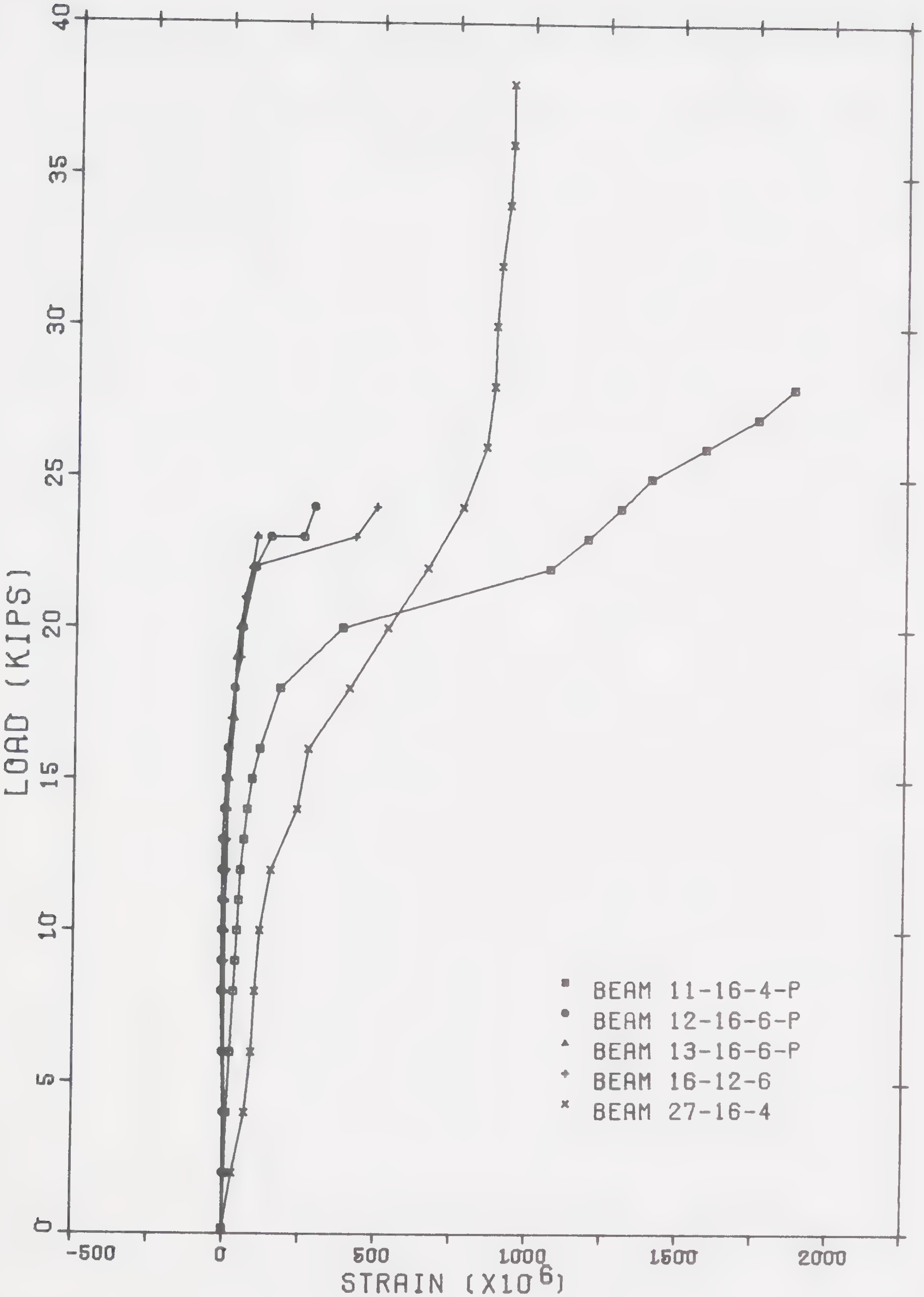


FIG. 4.64 LOAD VS. STRAIN FOR GAGE 14

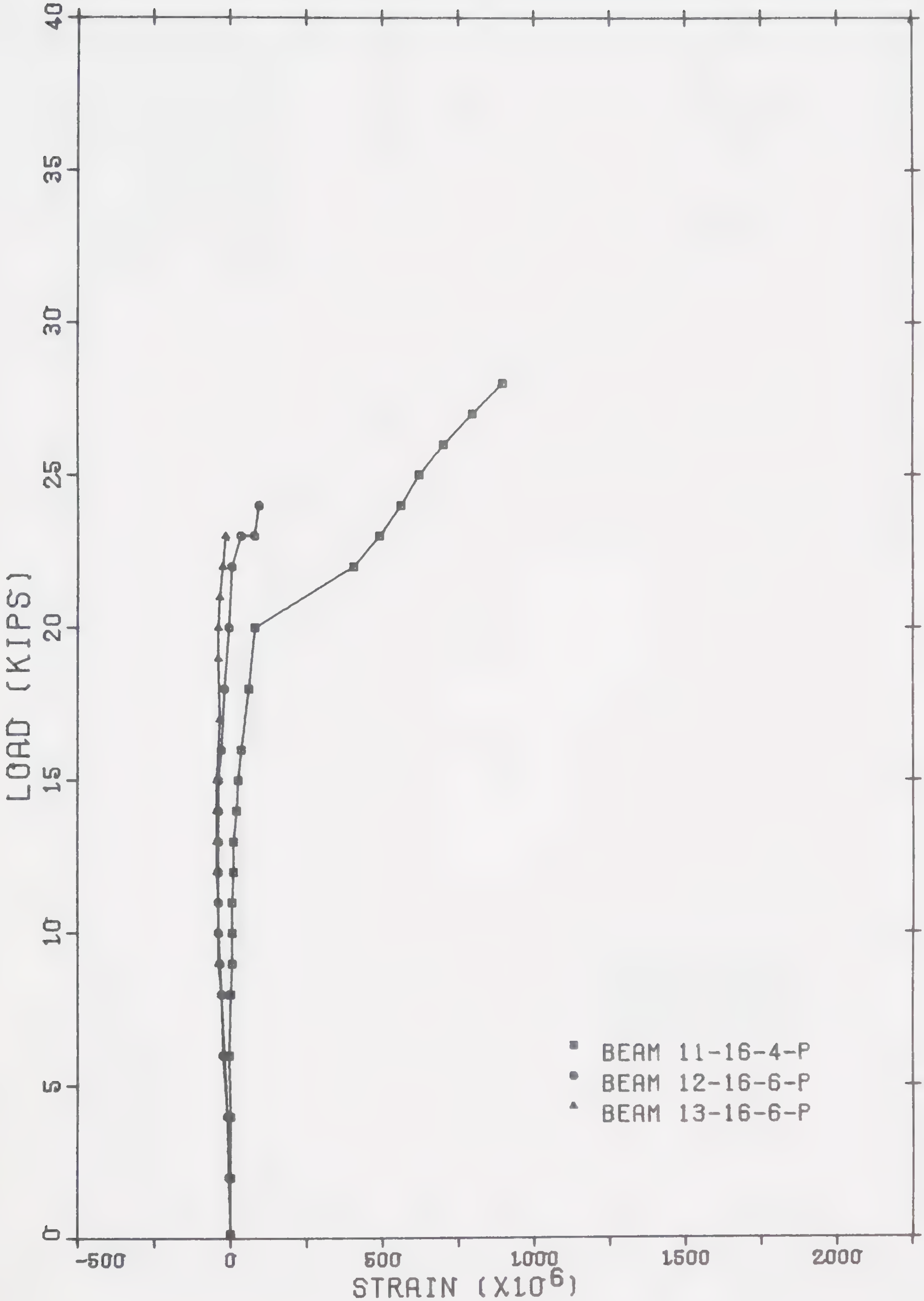


FIG. 4.65 LOAD VS. STRAIN FOR GAGE 15

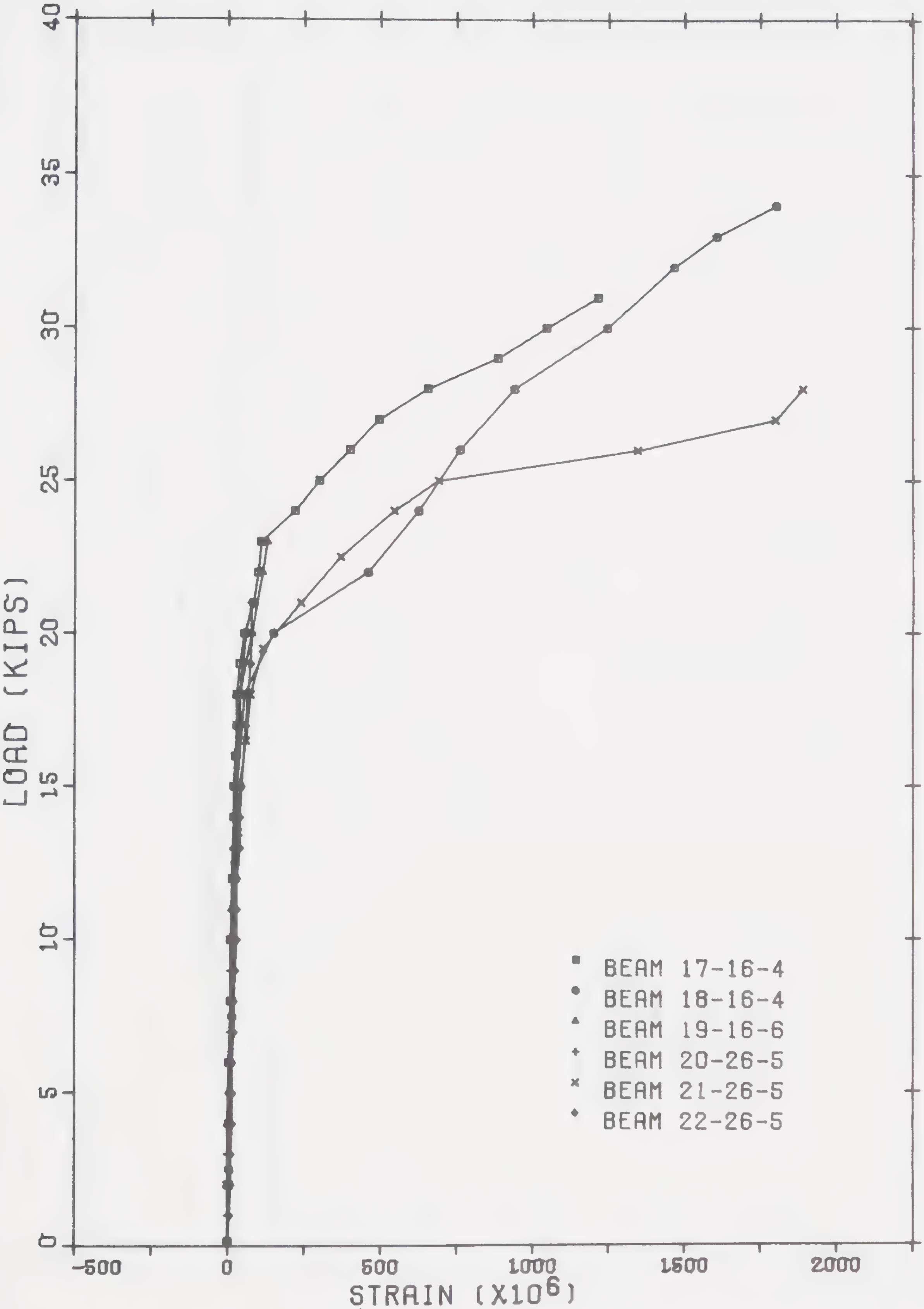


FIG. 4.66 LOAD VS. STRAIN FOR GAGE 15

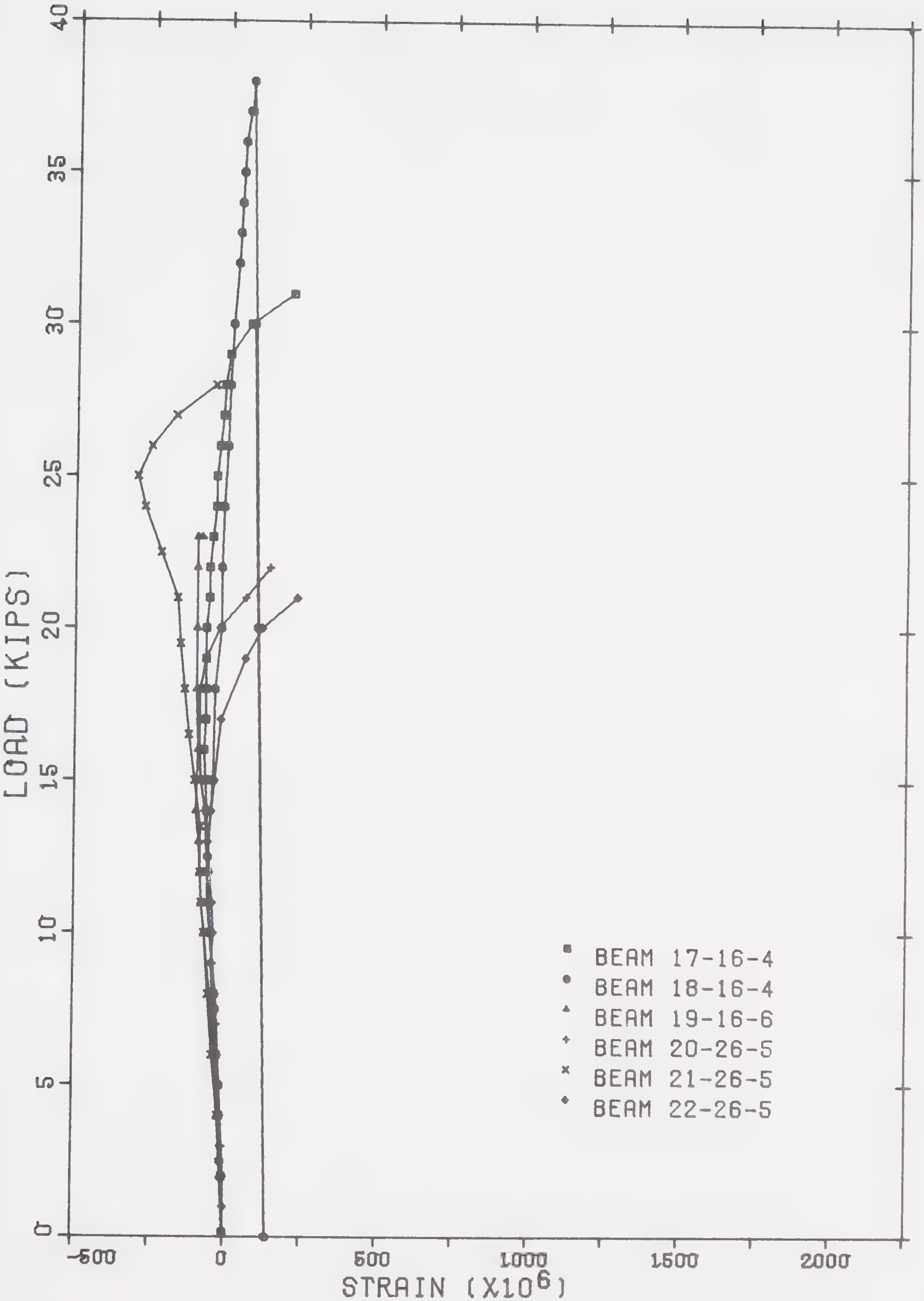
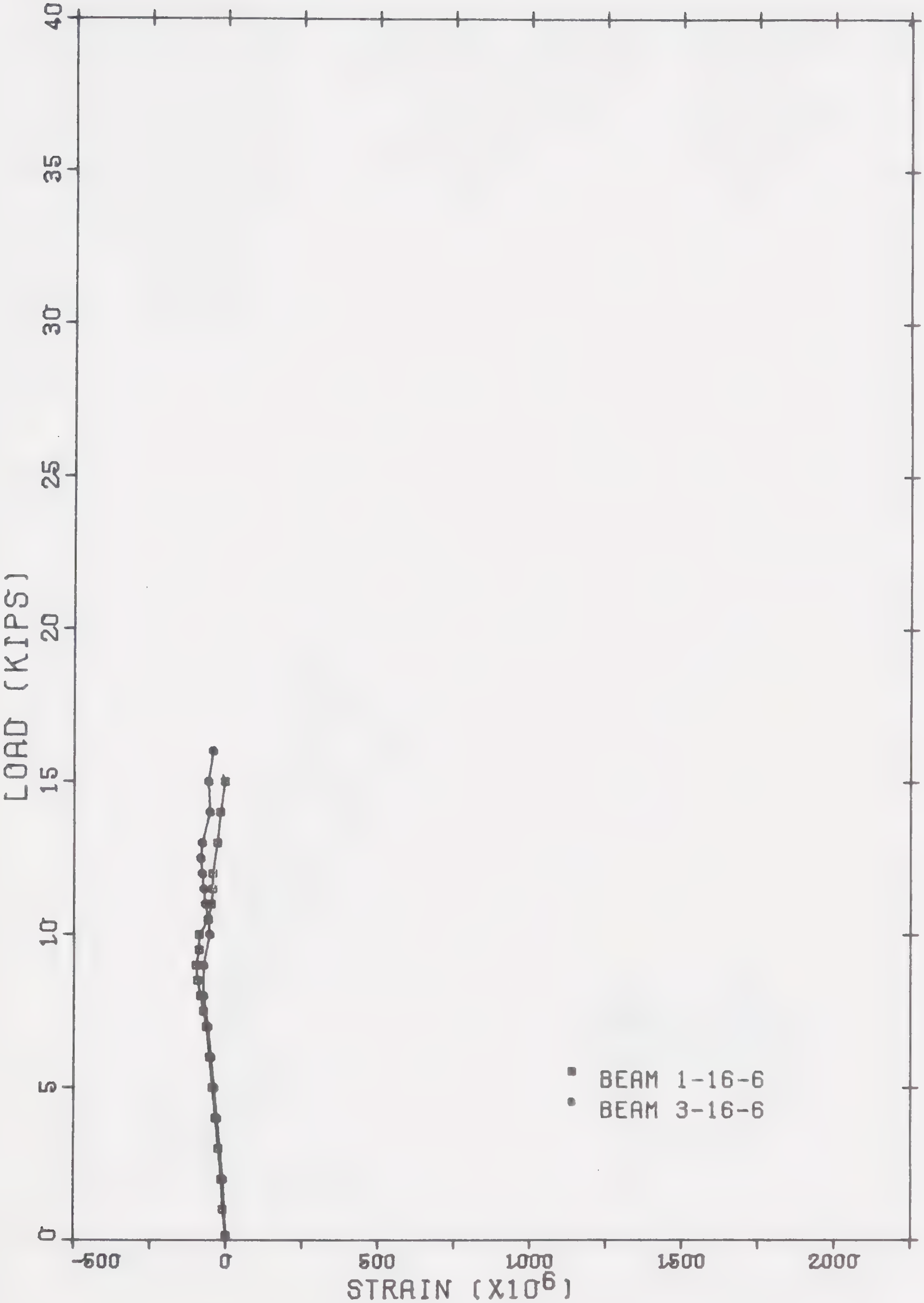


FIG. 4.67 LOAD VS. STRAIN FOR GAGE 16



EIG. 4.68 LOAD VS. STRAIN FOR GAGE 17

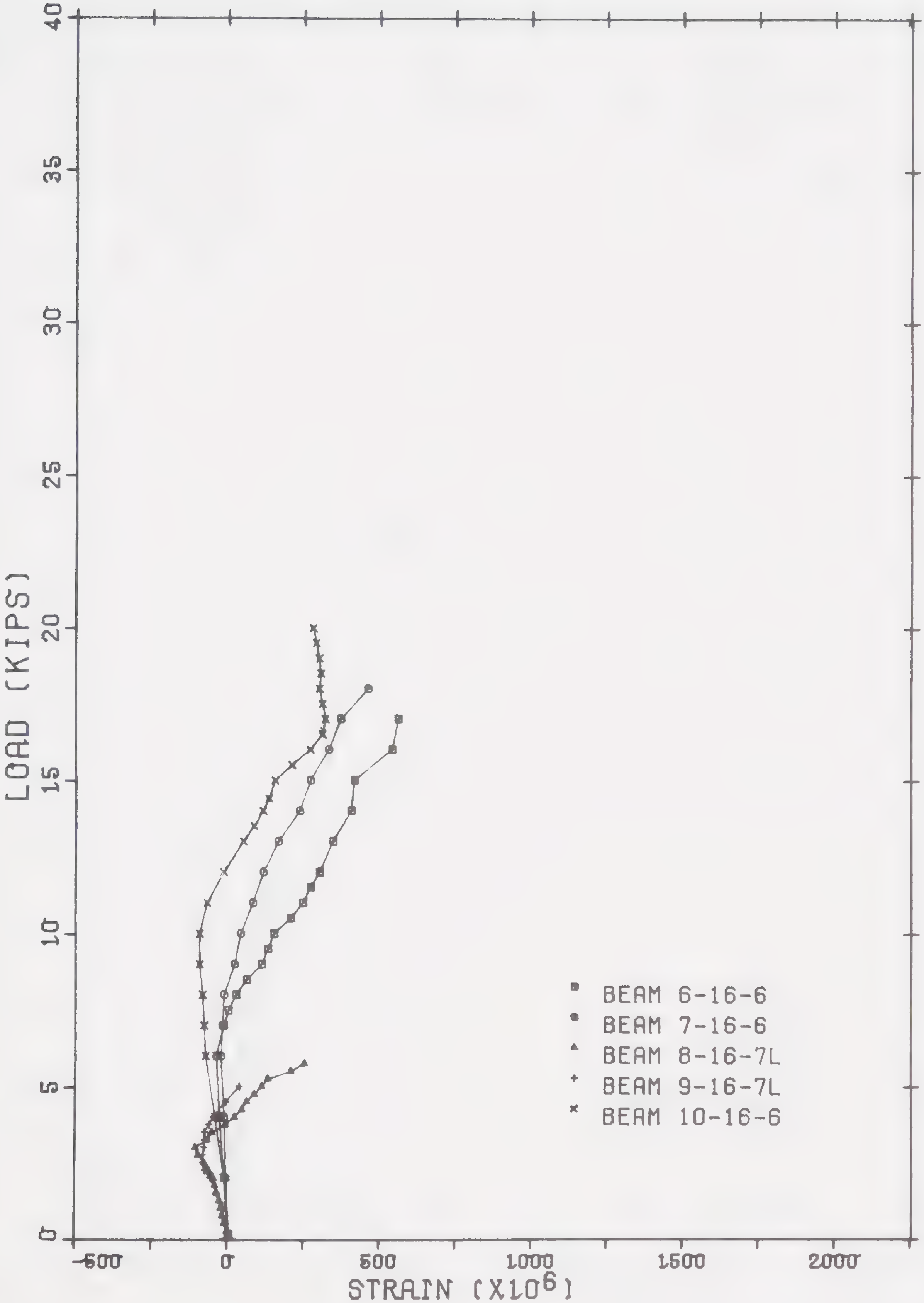


FIG. 4.69 LOAD VS. STRAIN FOR GAGE 17

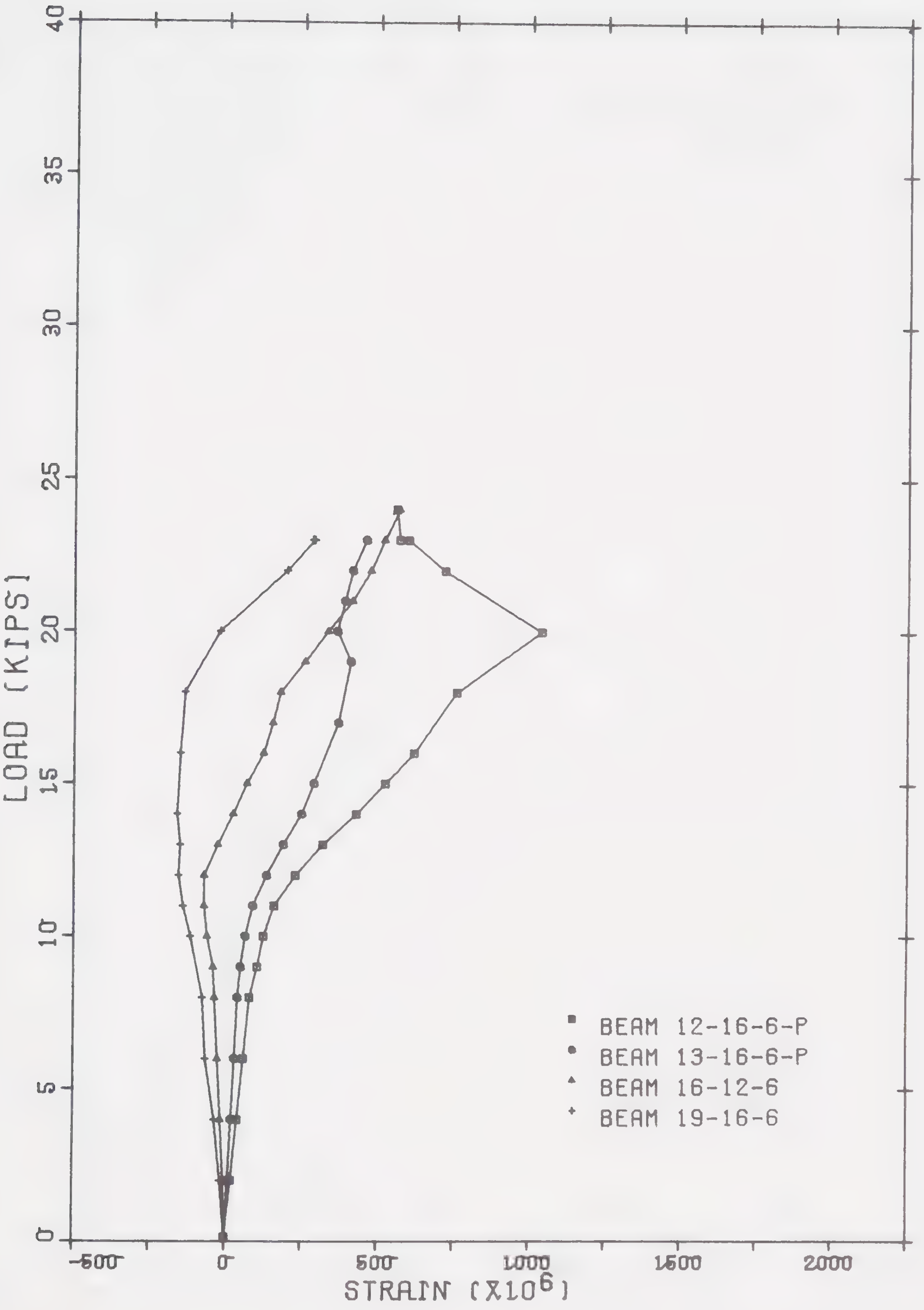


FIG. 4.70 LOAD VS. STRAIN FOR GAGE 17

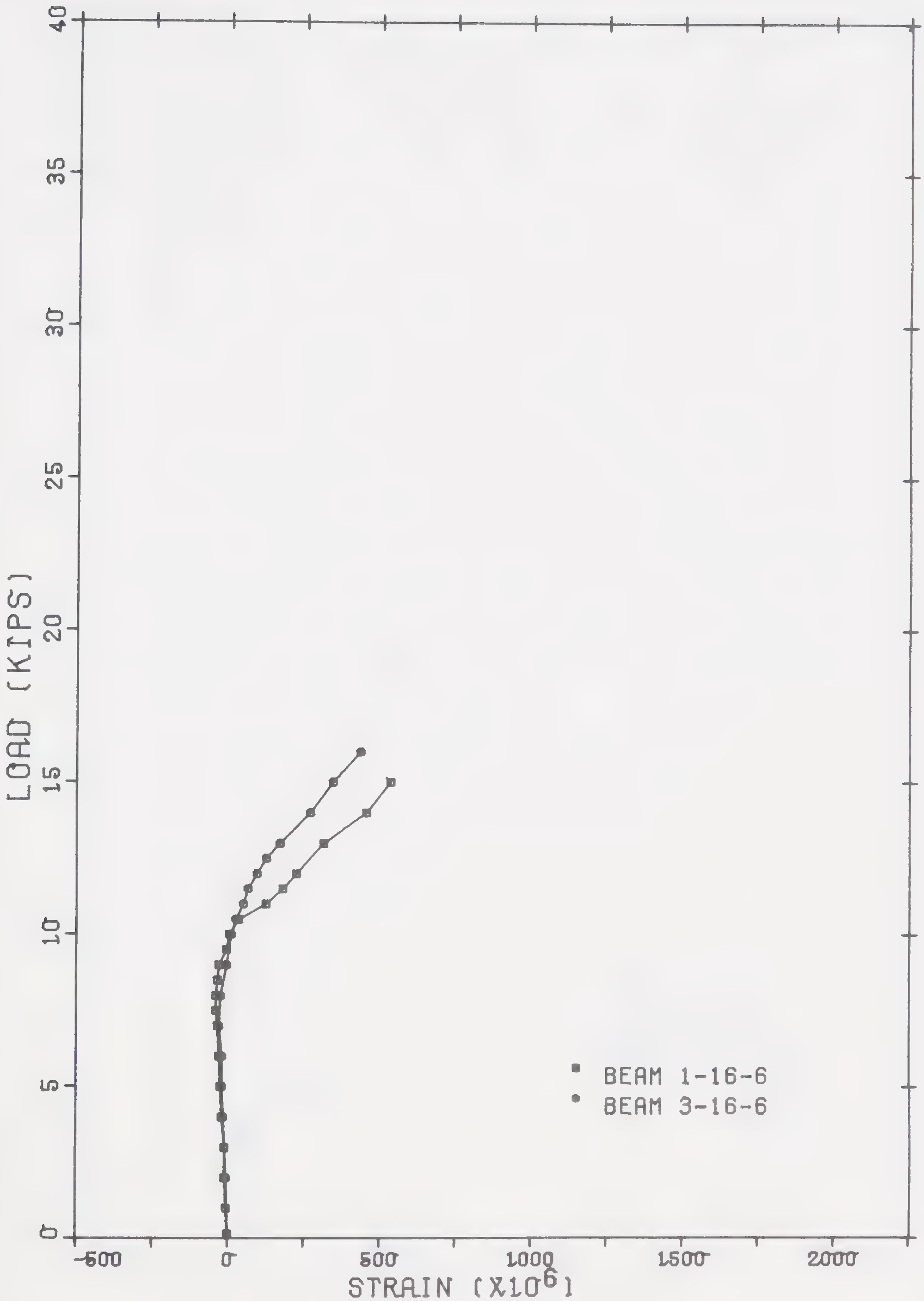


FIG. 4.71 LOAD VS. STRAIN FOR GAGE 18

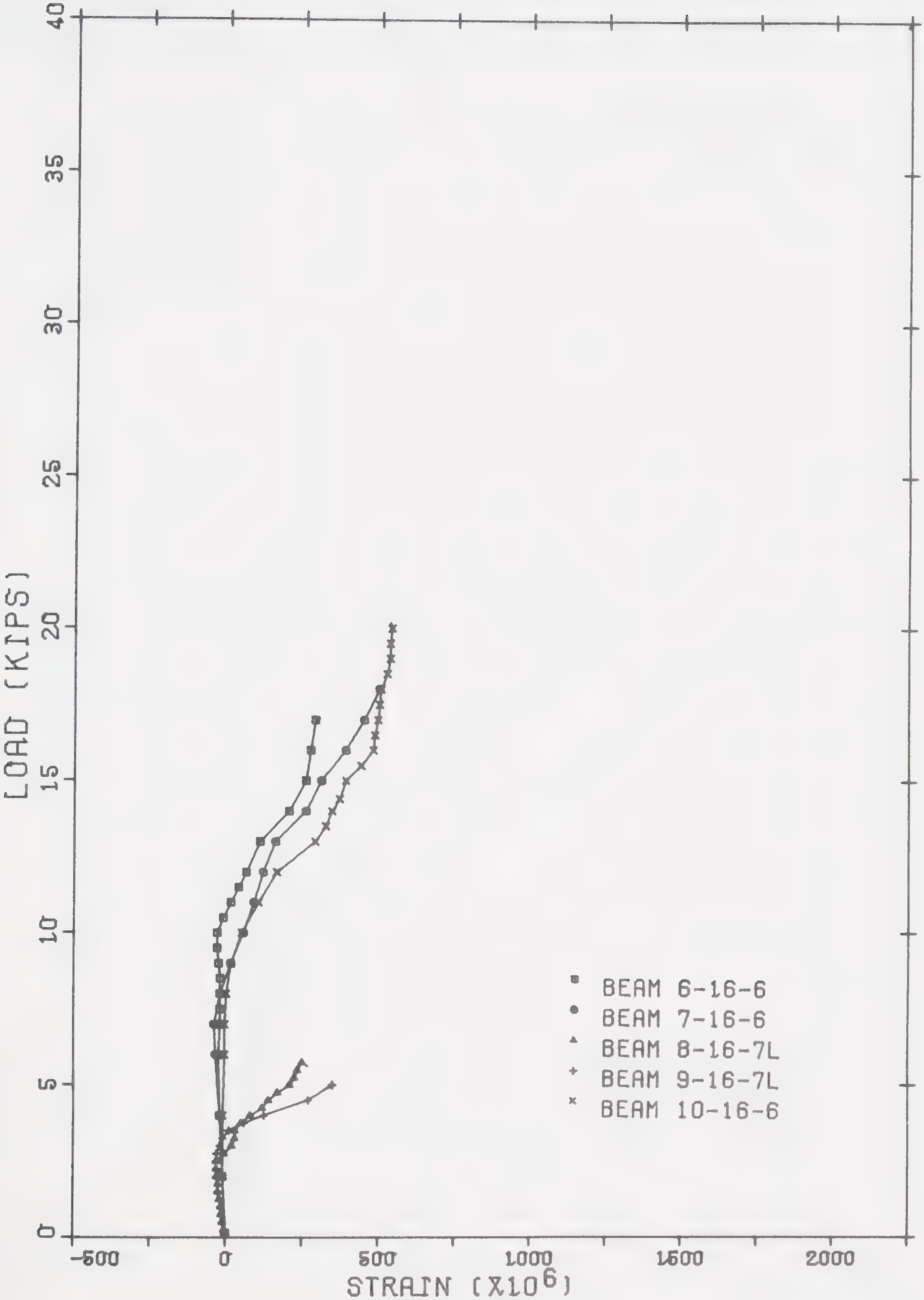


FIG. 4.72 LOAD VS. STRAIN FOR GAGE 18

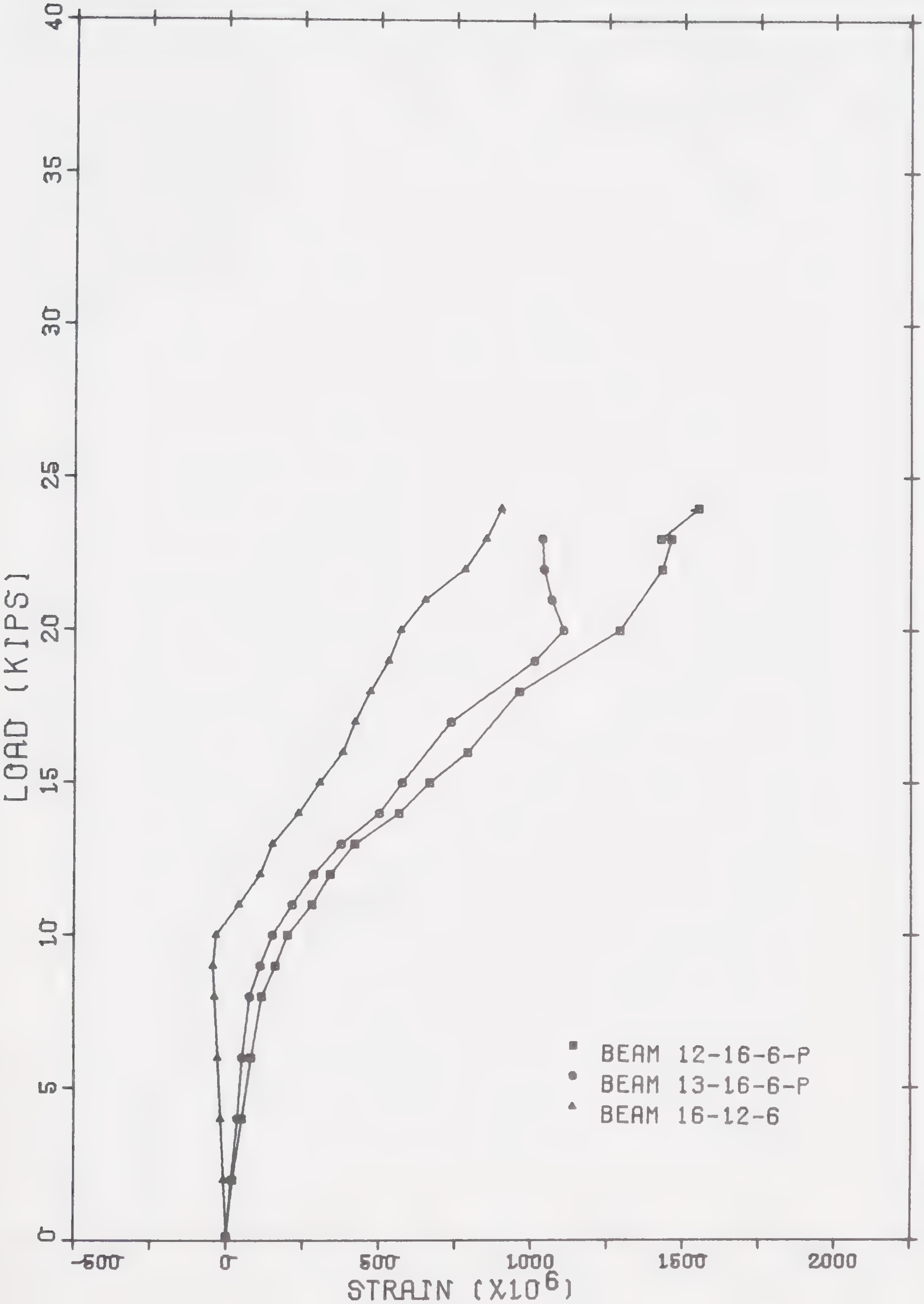


FIG. 4.73 LOAD VS. STRAIN FOR GAGE 18

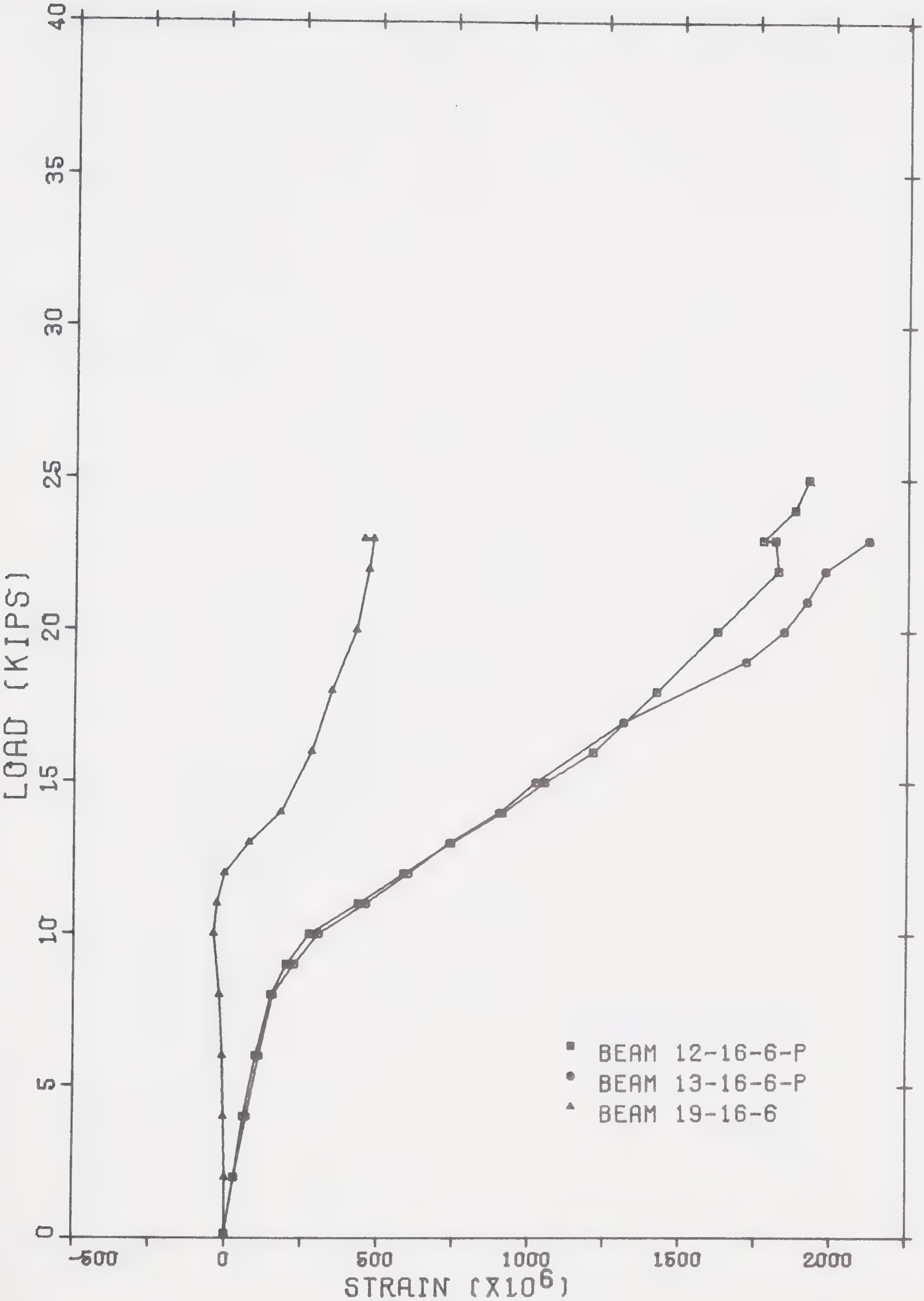


FIG. 4.74 LOAD VS. STRAIN FOR GAGE 19

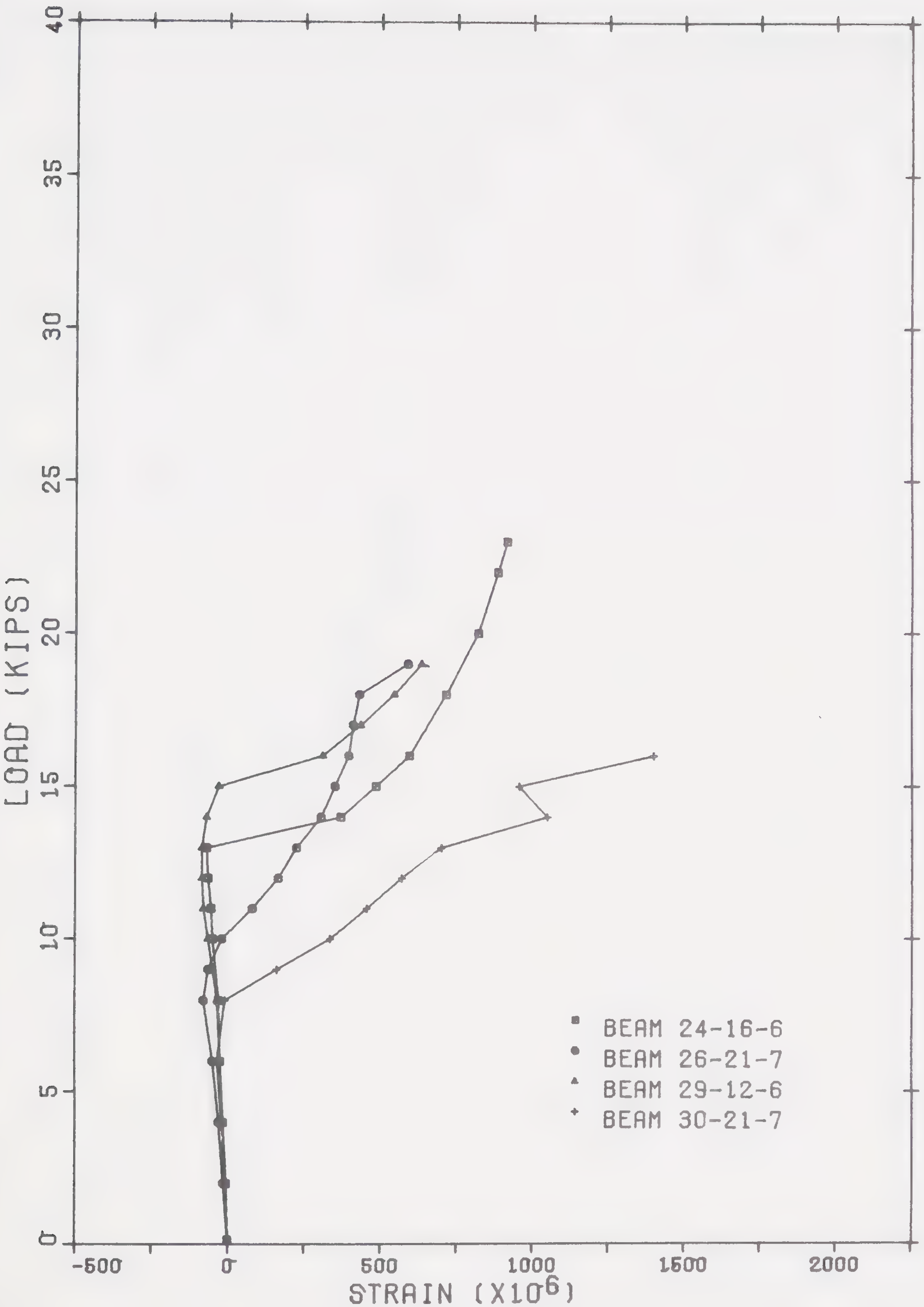


FIG. 4.75 LOAD VS. STRAIN FOR GAGE 19

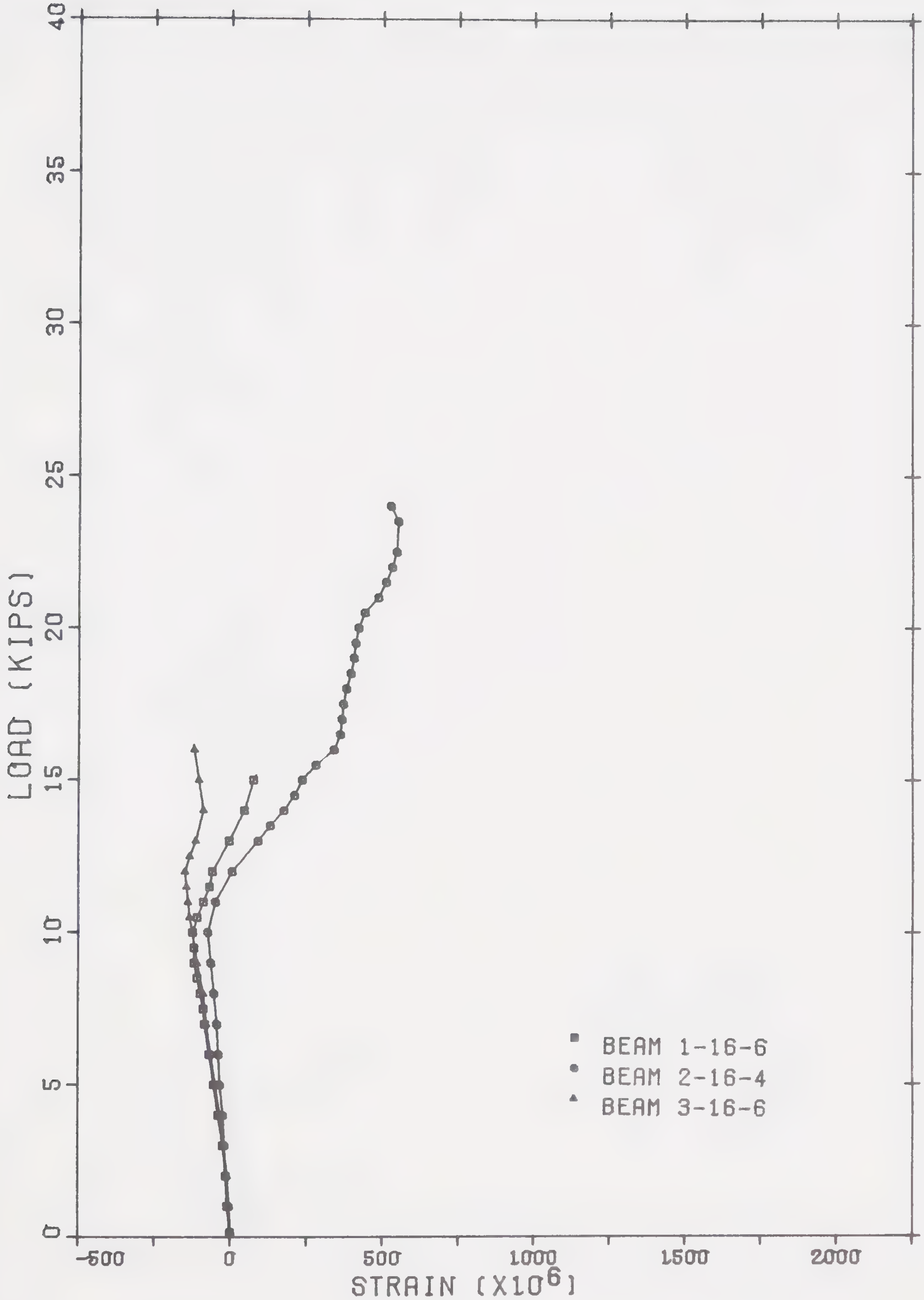


FIG. 4.76 LOAD VS. STRAIN FOR GAGE 21

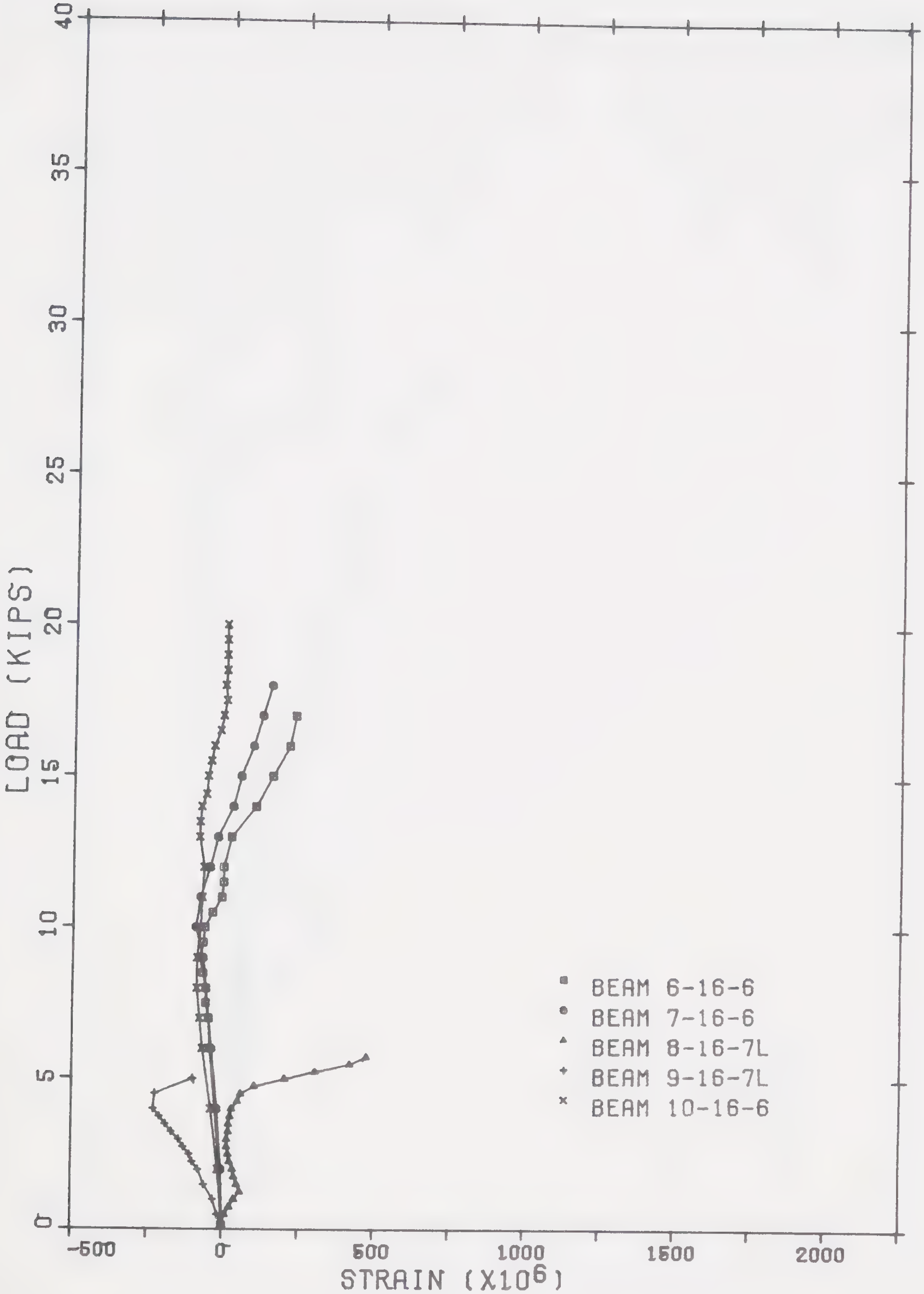


FIG. 4.77 LOAD VS. STRAIN FOR GAGE 21

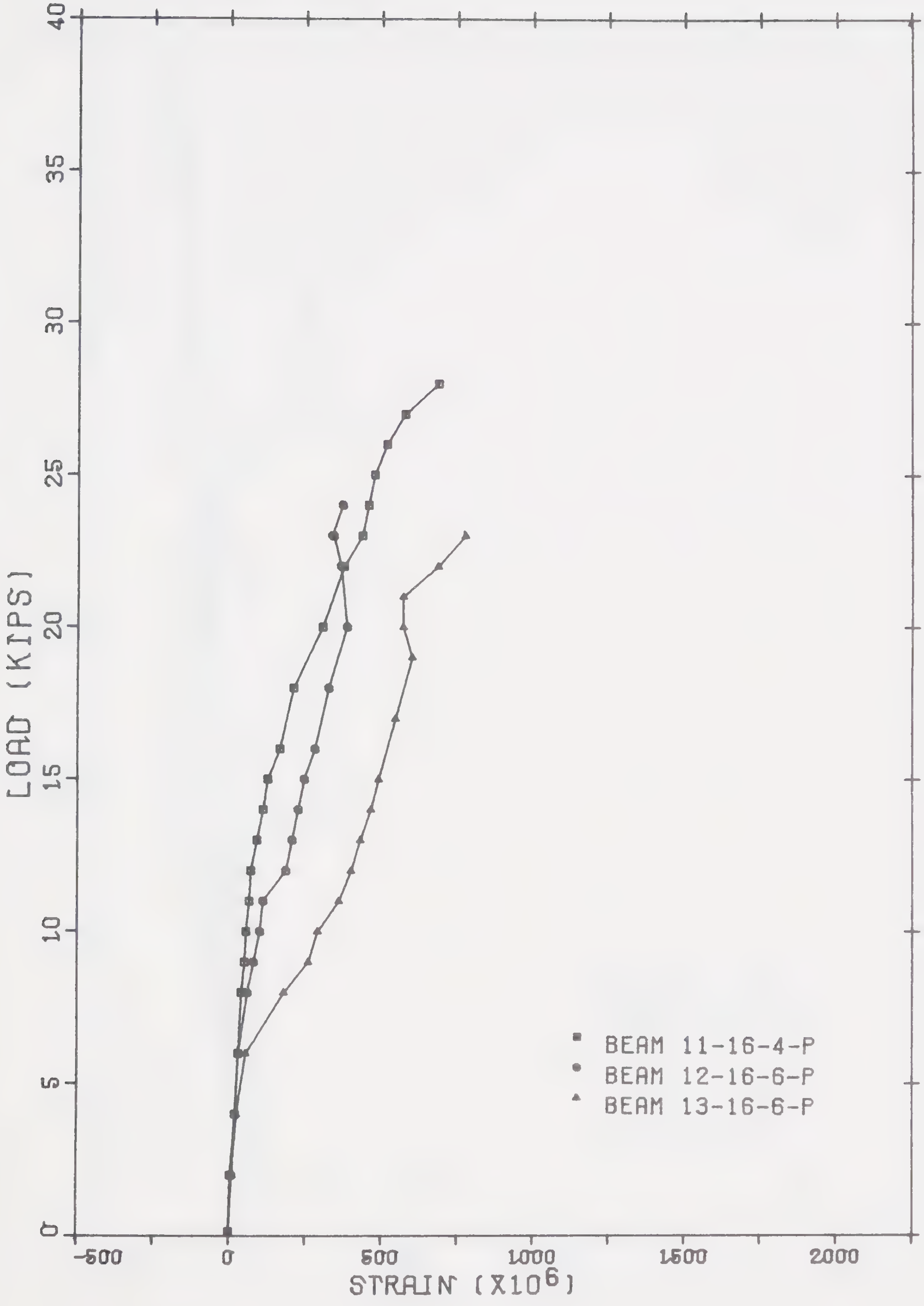


FIG. 4.78 LOAD VS. STRAIN FOR GAGE 21

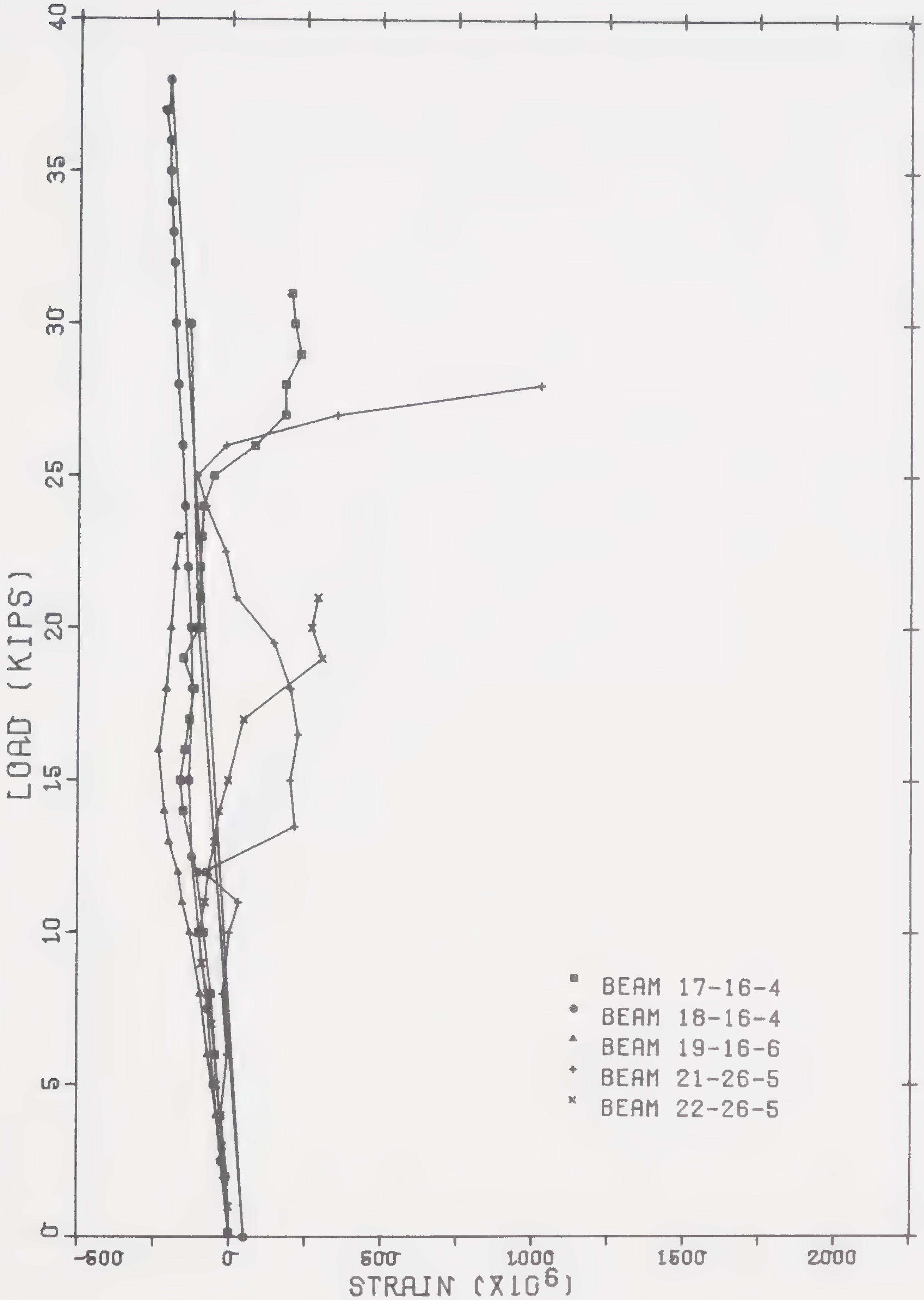


FIG. 4.79 LOAD VS. STRAIN FOR GAGE 21

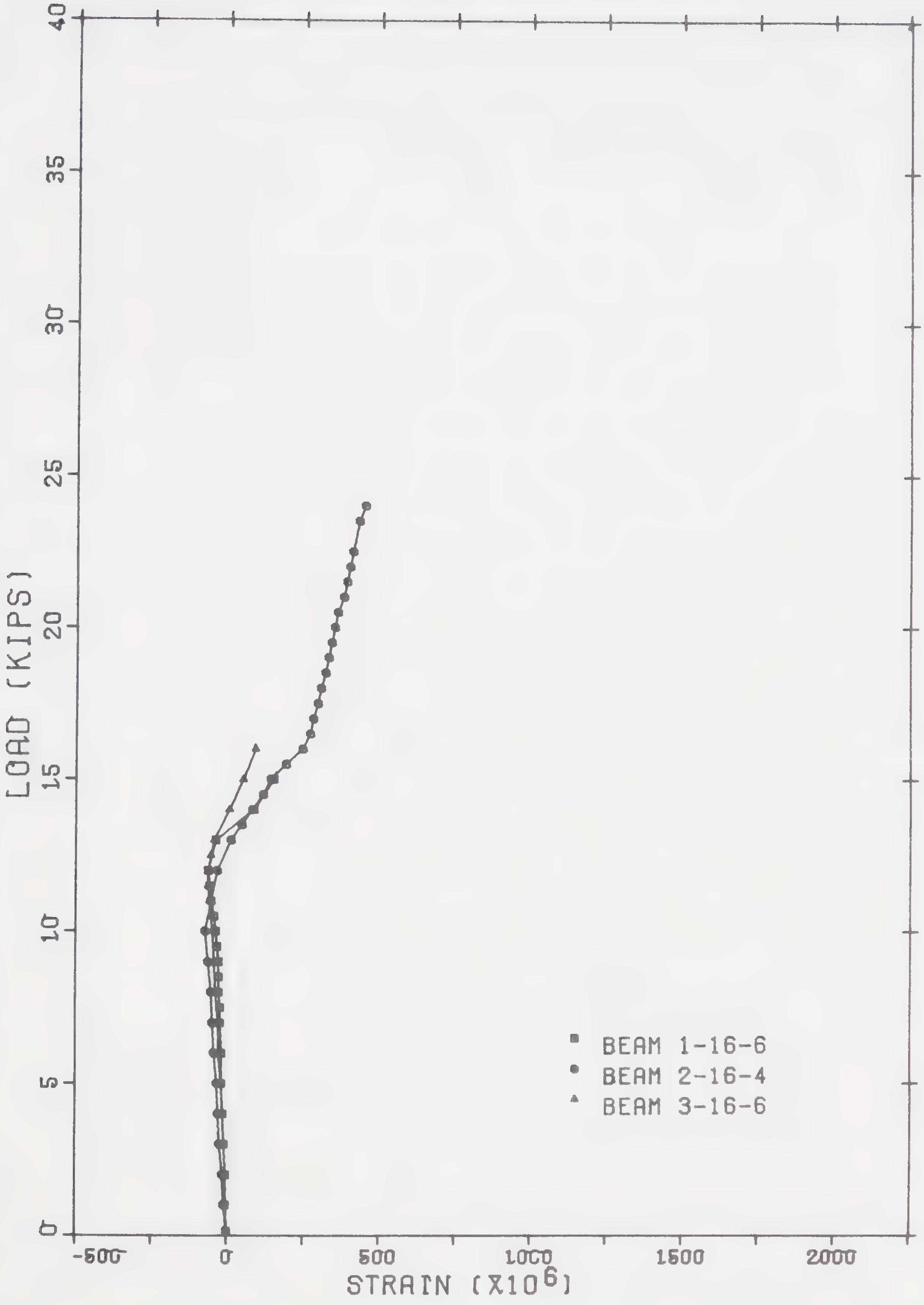


FIG. 4.80 LOAD VS. STRAIN FOR GAGE 22

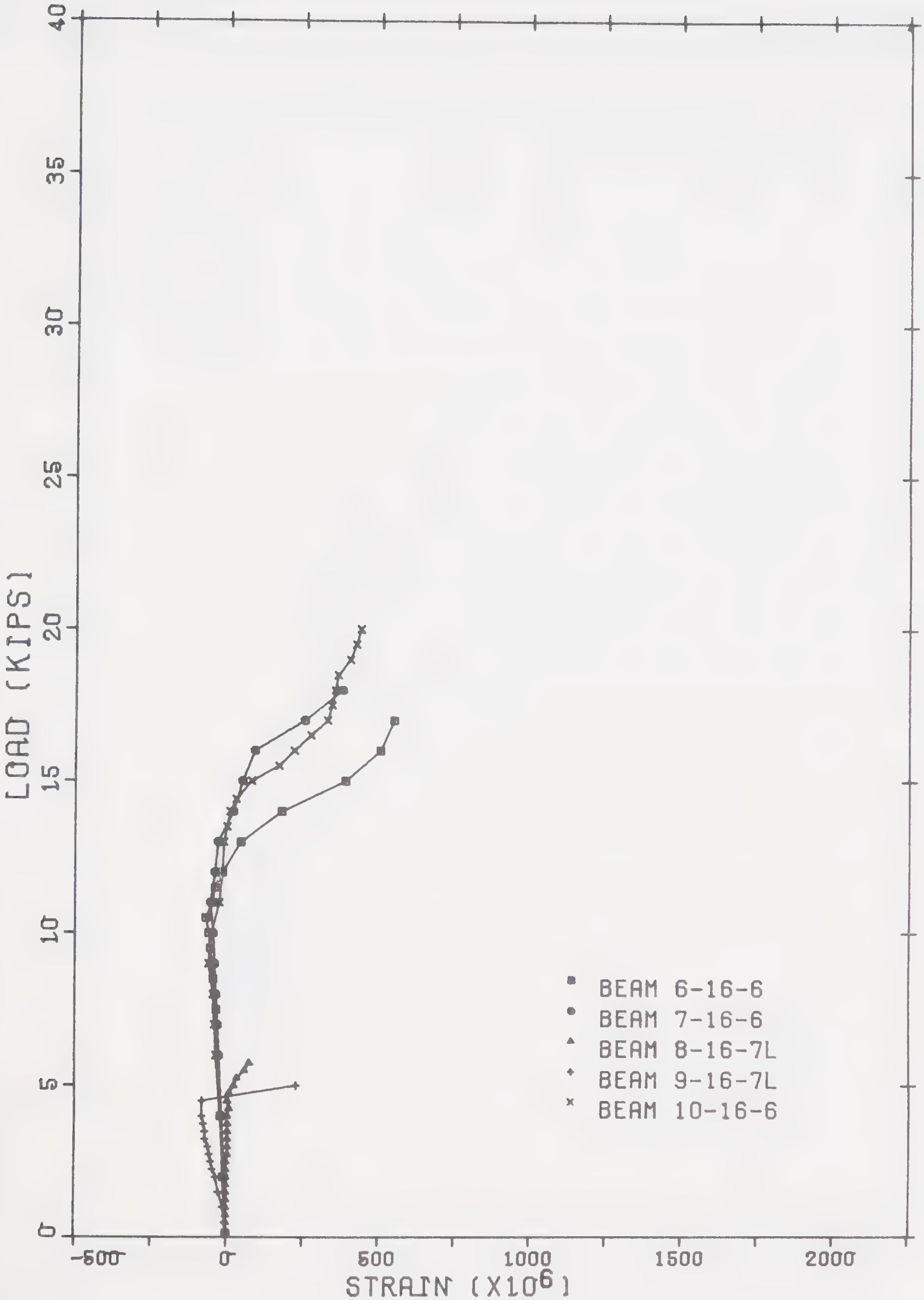


FIG. 4.81 LOAD VS. STRAIN FOR GAGE 22

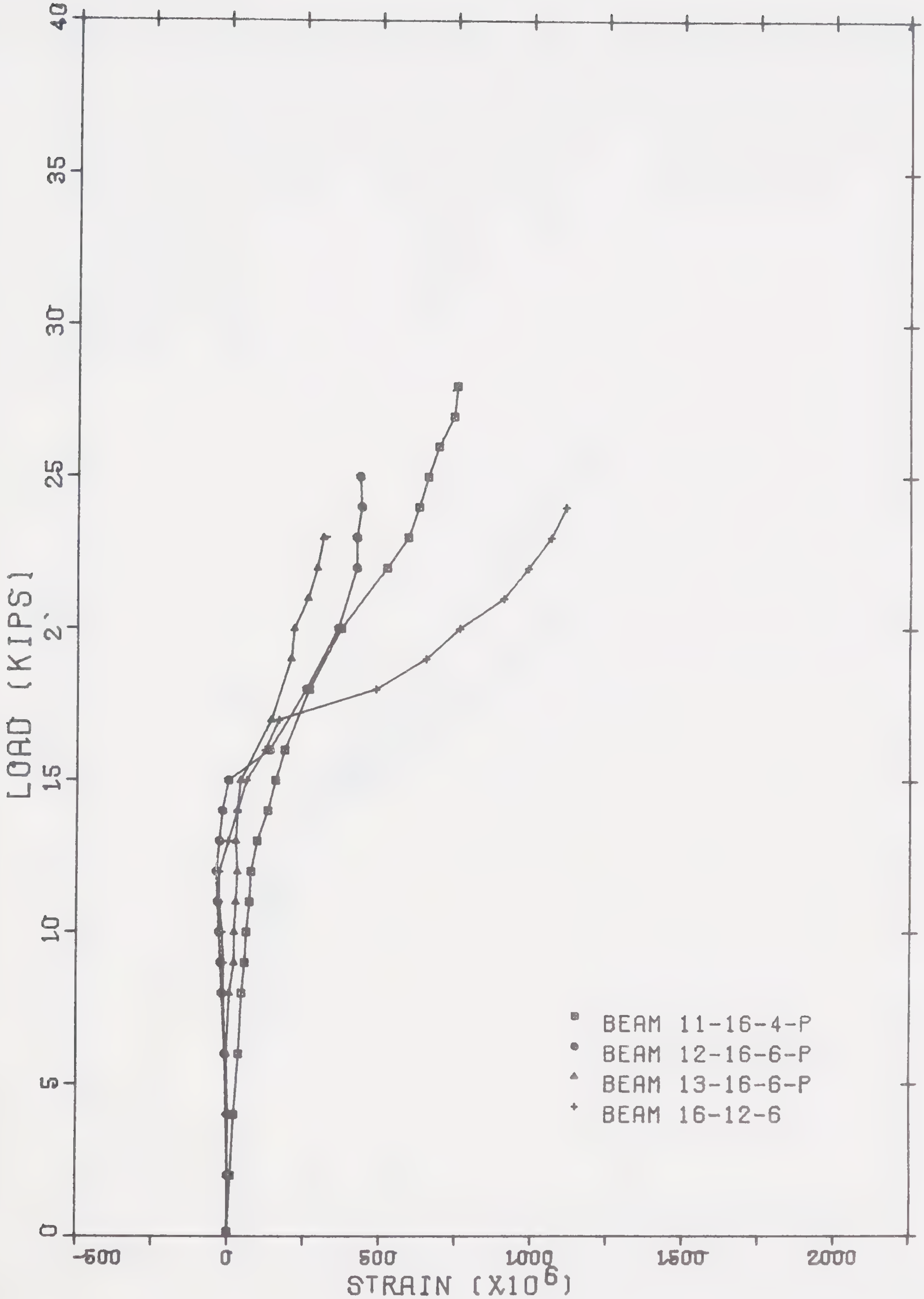


FIG. 4.82 LOAD VS. STRAIN FOR GAGE 22

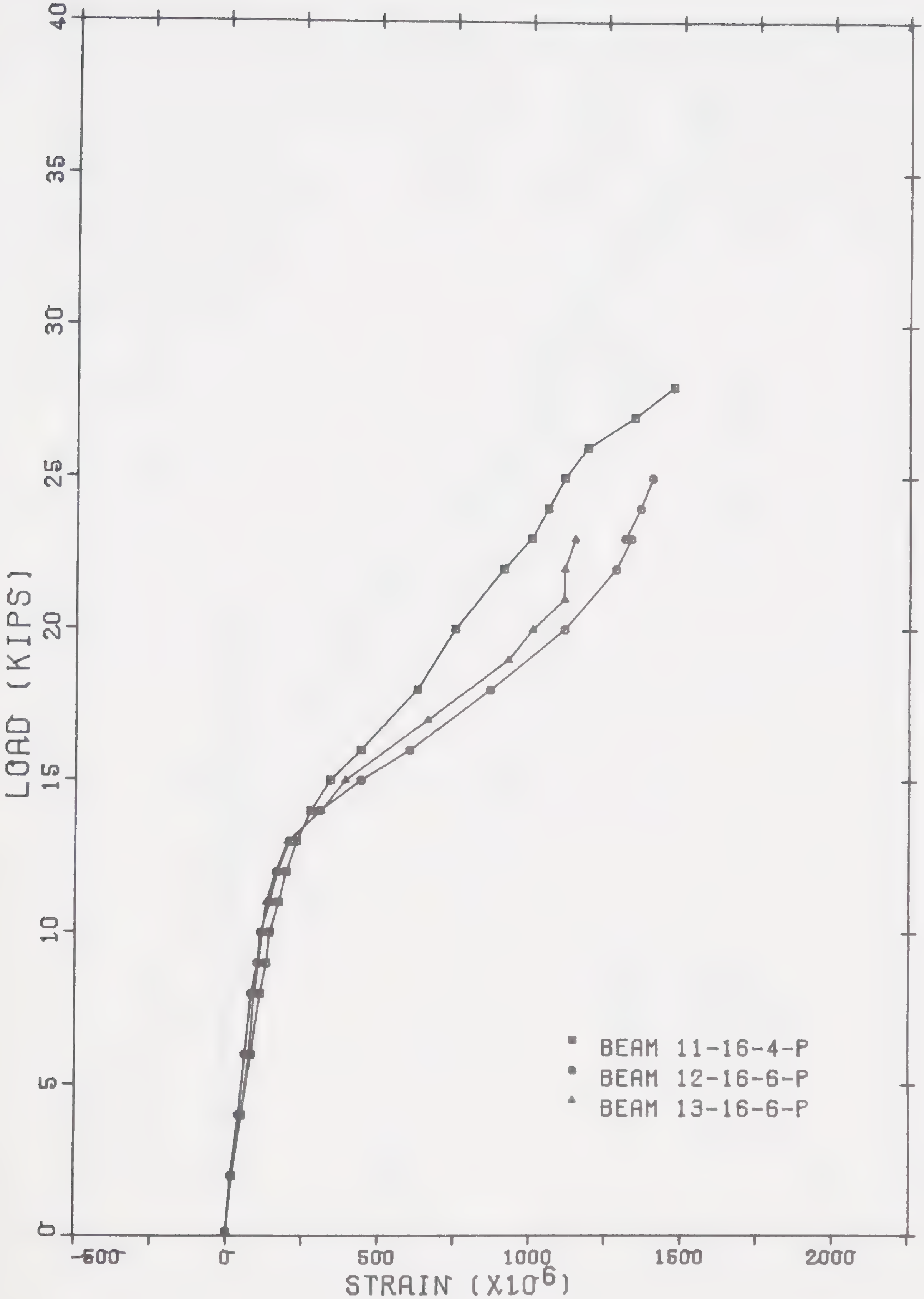
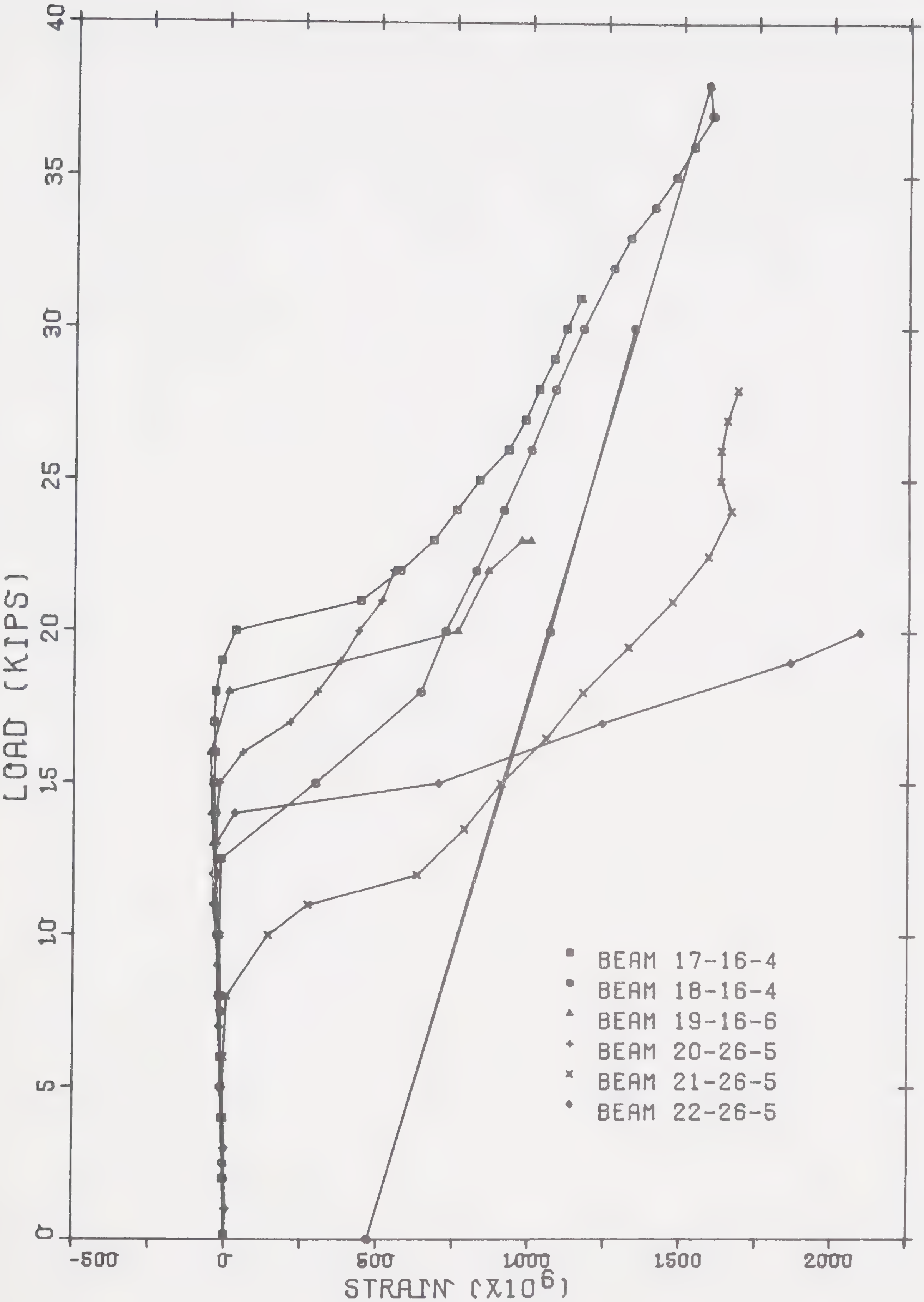


FIG. 4.83 LOAD VS. STRAIN FOR GAGE 23



EIG. 4.84 OAD VS. STRAIN FOR GAGE 23

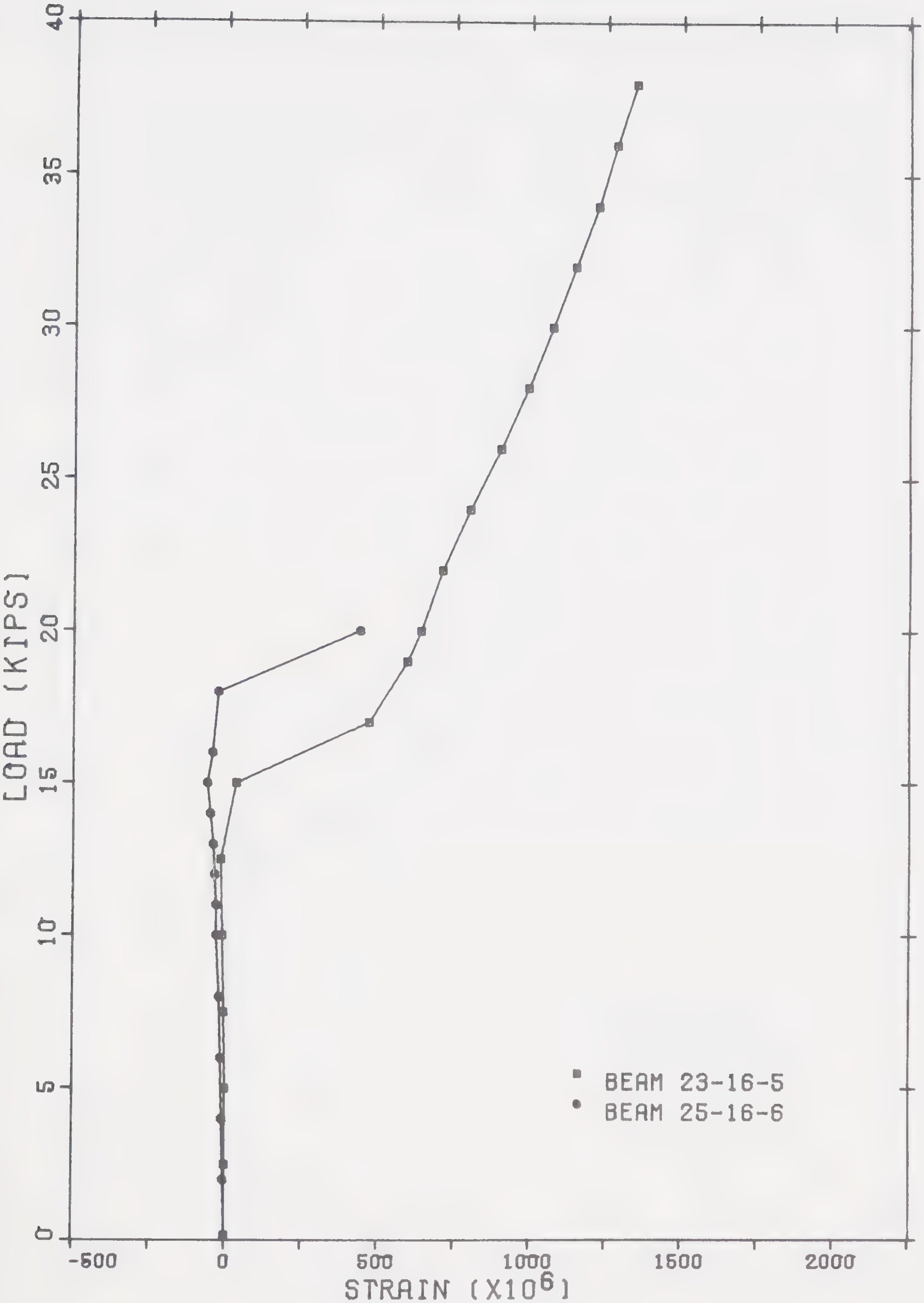


FIG. 4.85 LOAD VS. STRAIN FOR GAGE 23

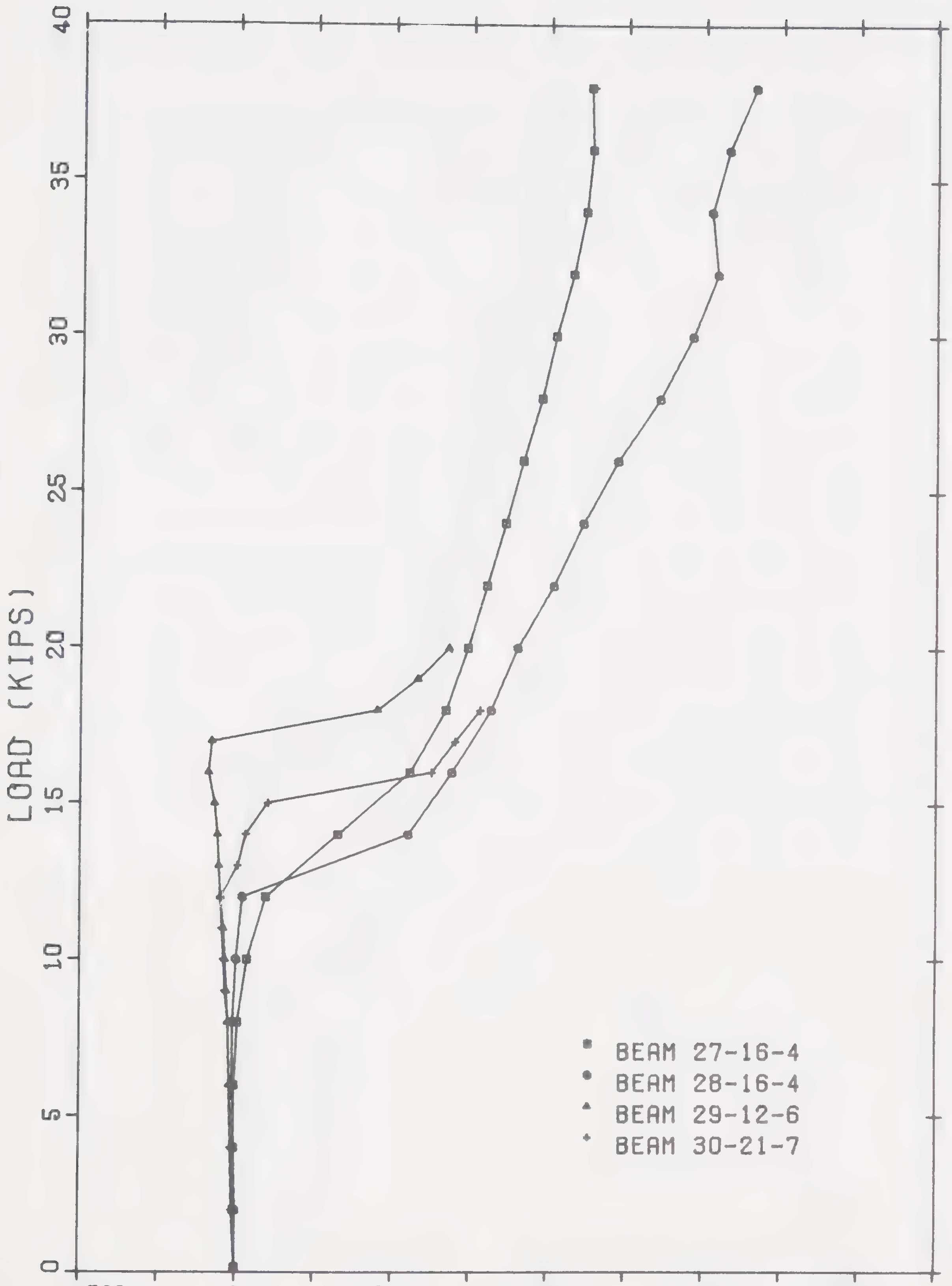


FIG. 4.86 LOAD VS. STRAIN FOR GAGE 23

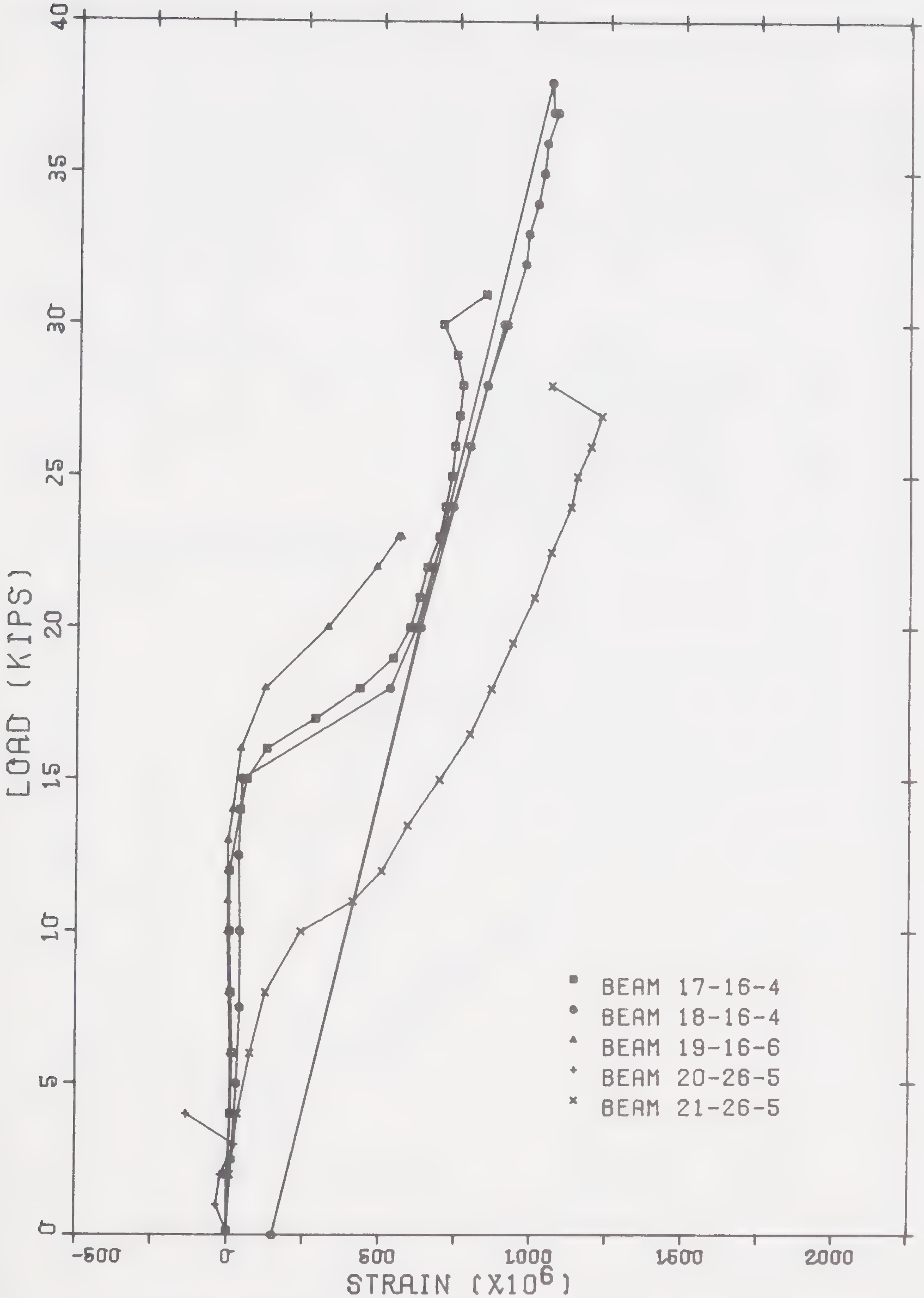


FIG. 4.87 LOAD VS. STRAIN FOR GAGE 24

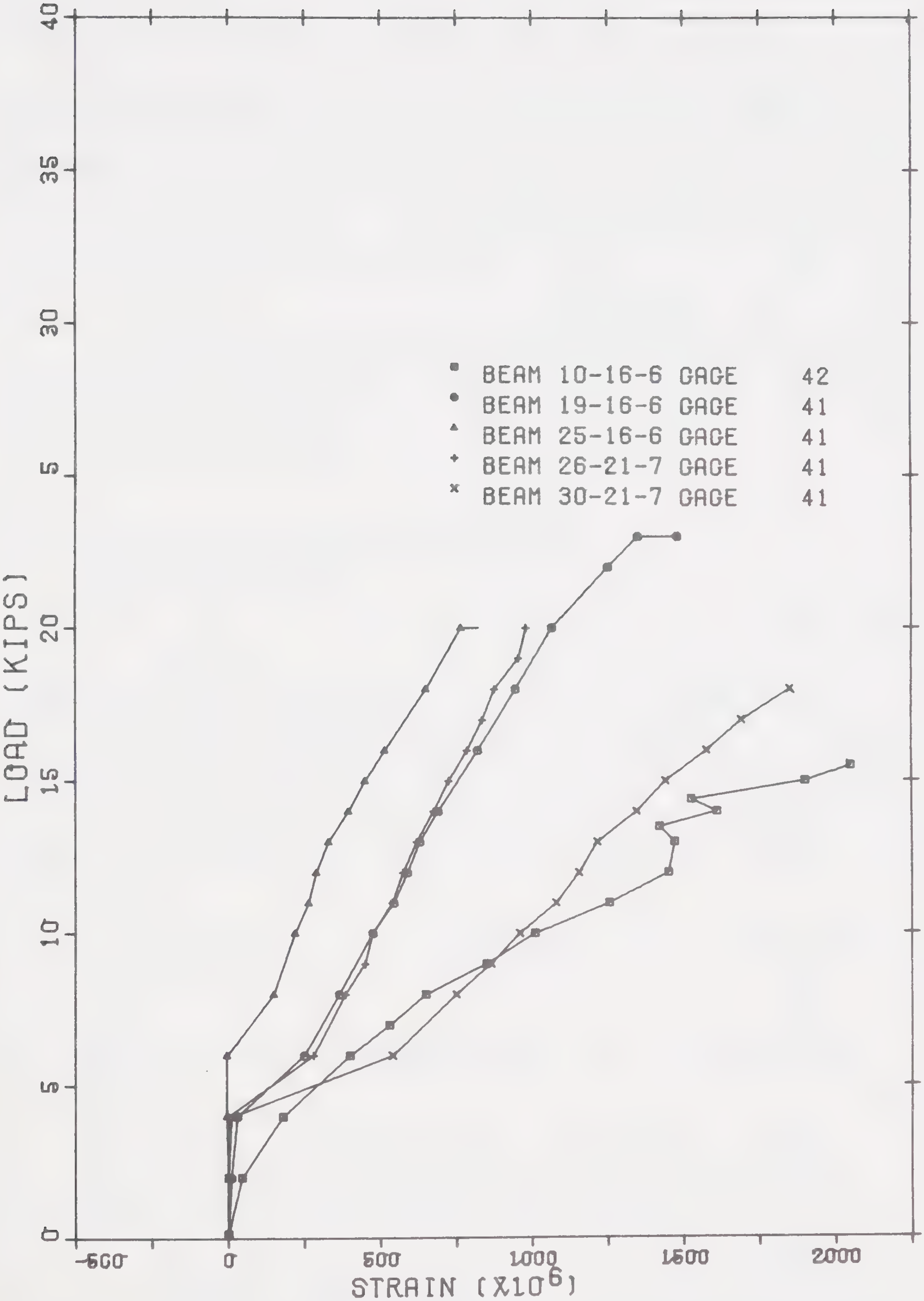


FIG. 4.88 LOAD VS. STRAIN FOR GAGES 41 AND

are plotted in Figures 4.48 to 4.67. The strain in the bottom strut stirrups at gage locations 17 to 24 are plotted in Figures 4.68 to 4.87. The strain in the supplementary post reinforcement at gage locations 41 and 42 are plotted in Figure 4.88.

Two hundred and seventy-one gages were mounted on the shear reinforcement for all the beams; 4% were inoperative at the time of testing and are not plotted.

4.6 Load Strain Relationships for the Strut Flexural Reinforcement

Figures 4.89 to 4.102 are plots of load versus strain in the longitudinal strut reinforcement at gage locations 25 to 40. The strains were read directly from the electrical resistance strain gages and the loads per jack were read from the scales of the Amsler loading apparatus. The plots are arranged in four groups with four gages in each group:

1. Figures 4.89 and 4.90 for gages 25 to 28 in the top strut above hole 2.
2. Figures 4.91 to 4.95 for gages 29 to 32 in the top strut above hole 1.
3. Figures 4.96 and 4.97 for gages 33 to 36 below hole 2.
4. Figures 4.98 to 4.102 for gages 37 to 40 below hole 1.

Of the one hundred gages placed in all beams on the longitudinal strut reinforcement, 1% were inoperative at the time of testing and are not plotted.

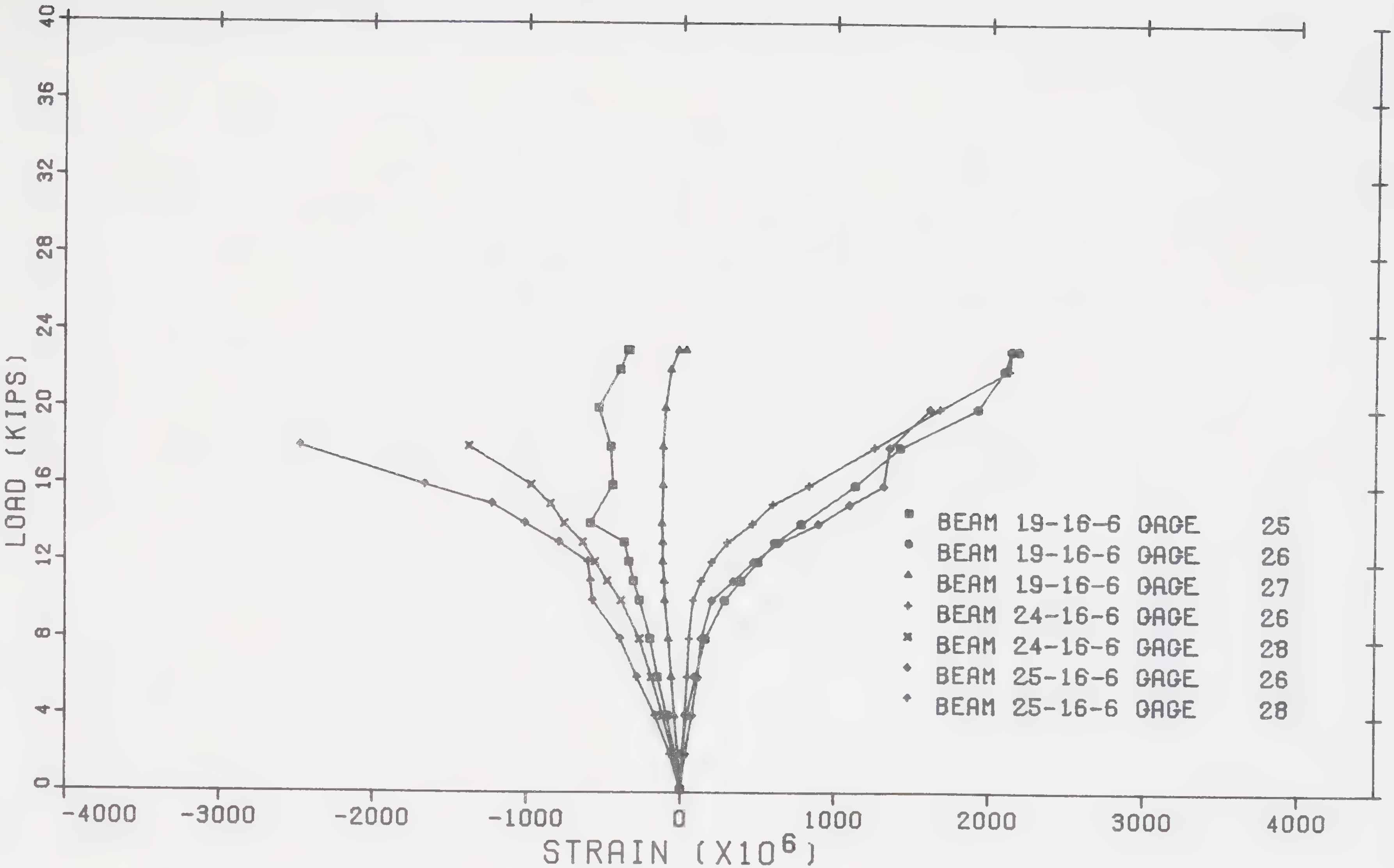


FIG. 4.89 LOAD VS. STRAIN FOR GAGES 25 TO 28

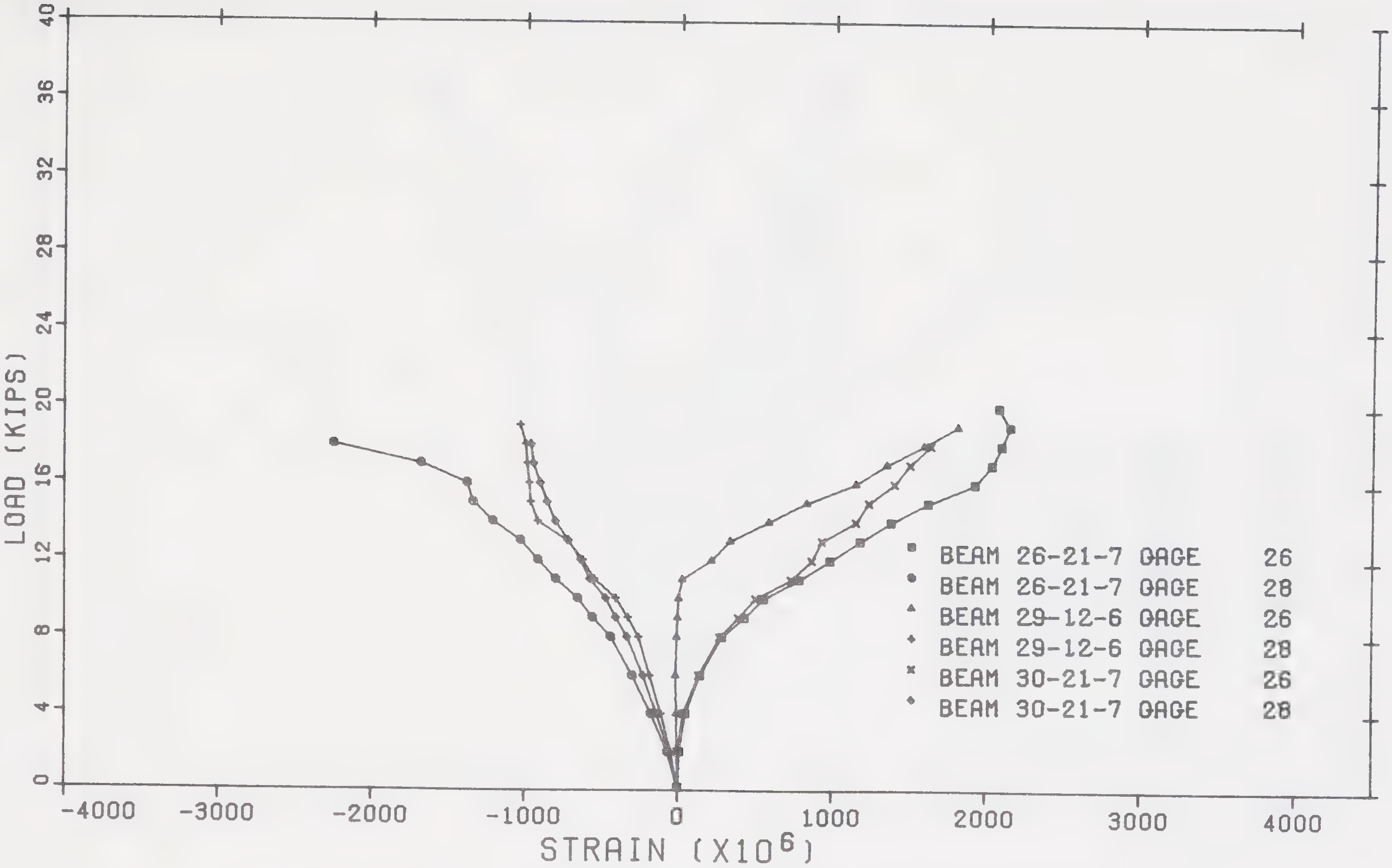


FIG. 4.90 LOAD VS. STRAIN FOR GAGES 25 TO 28

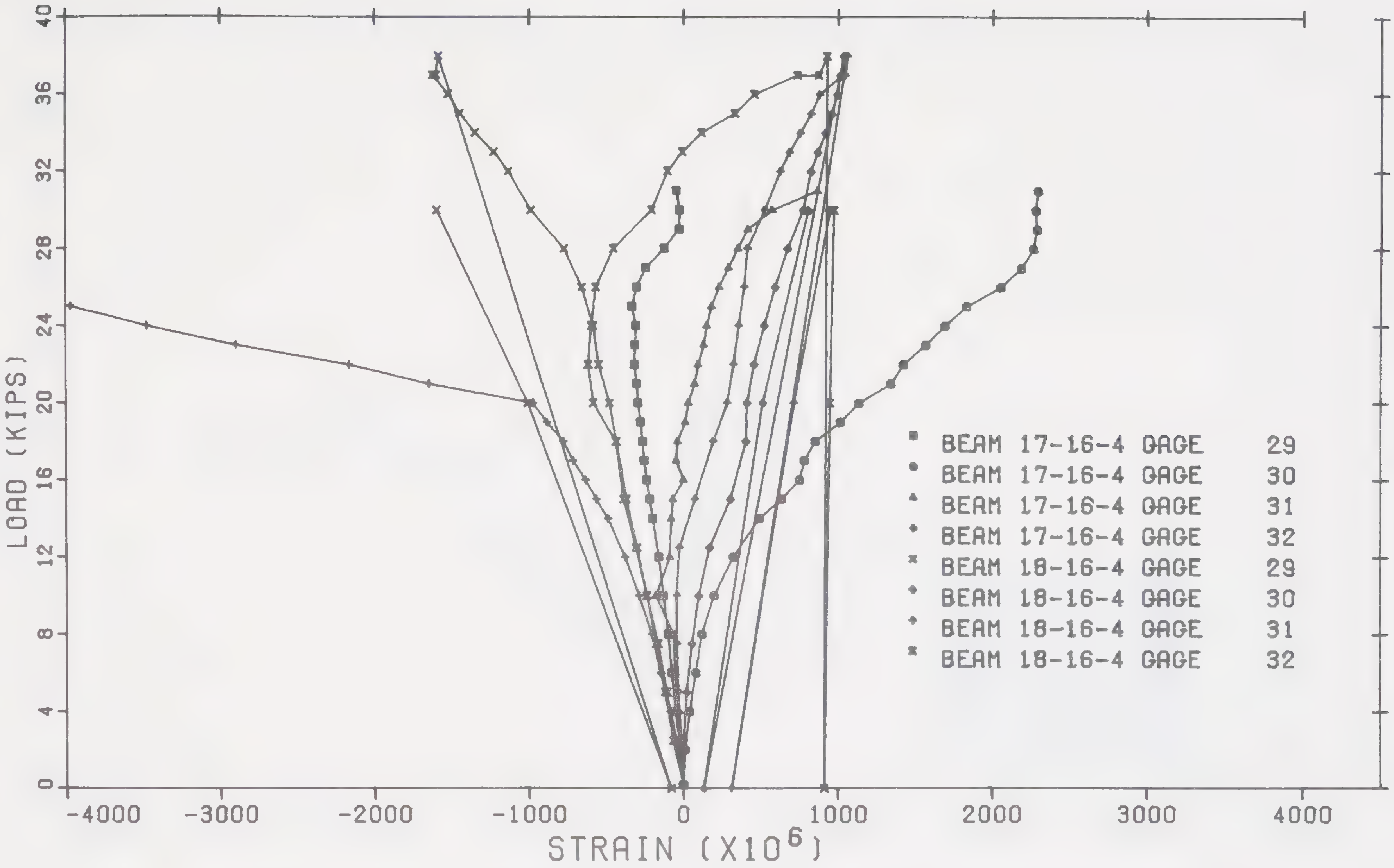


FIG. 4.91 LOAD VS. STRAIN FOR GAGES 29 TO 32

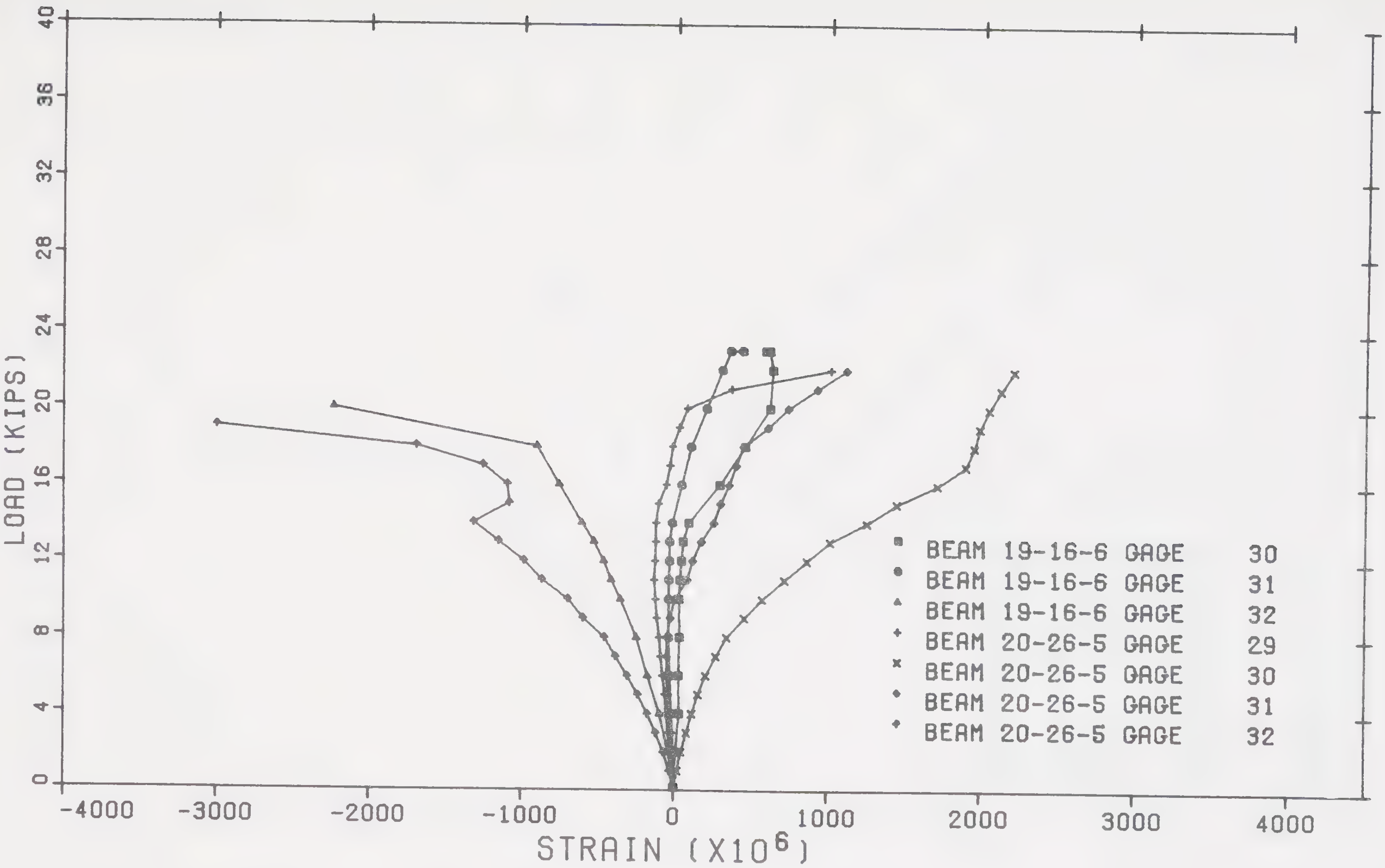


FIG. 4.92 LOAD VS. STRAIN FOR GAGES 29 TO 32

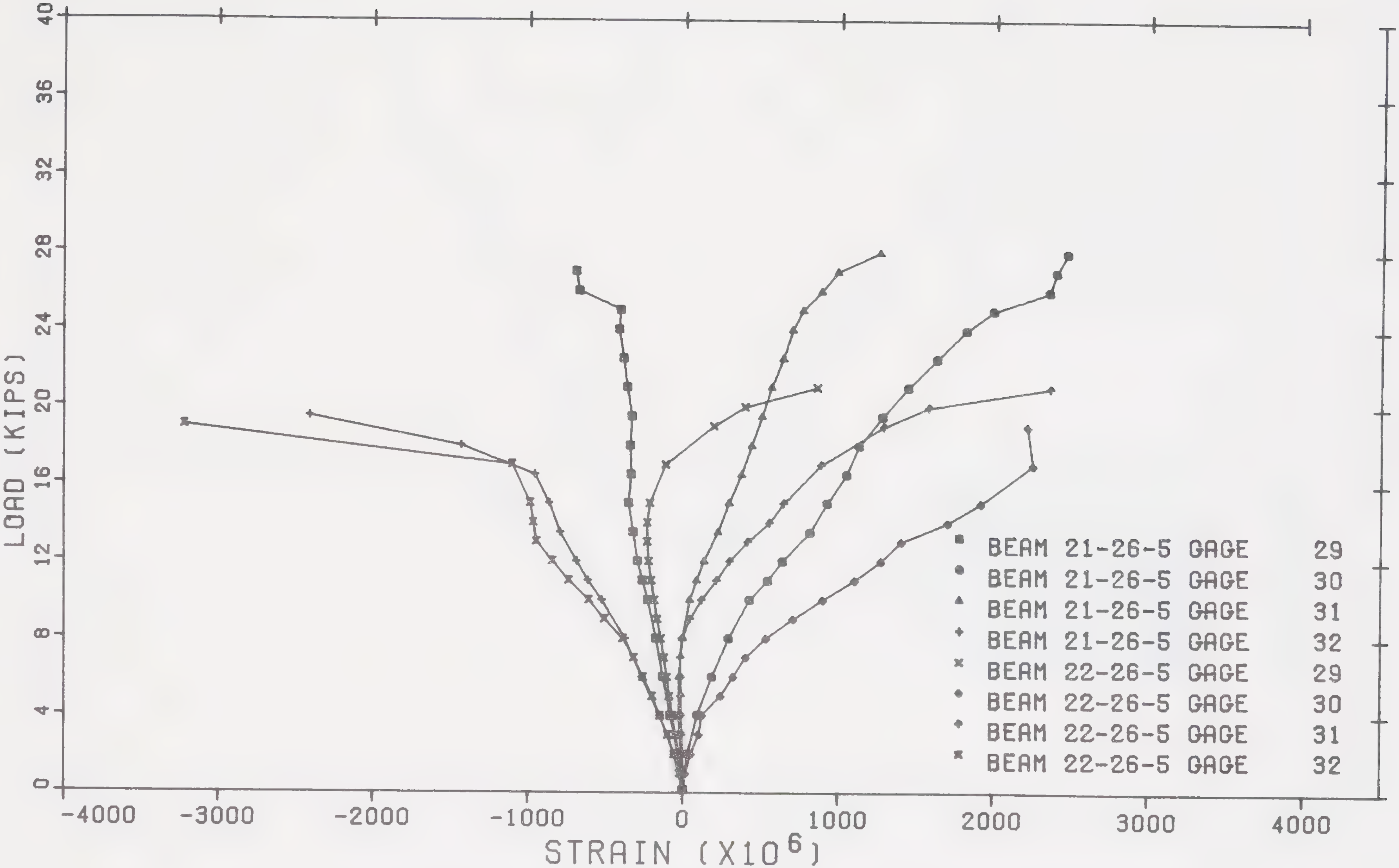


FIG. 4.93 LOAD VS. STRAIN FOR GAGES 29 TO 32

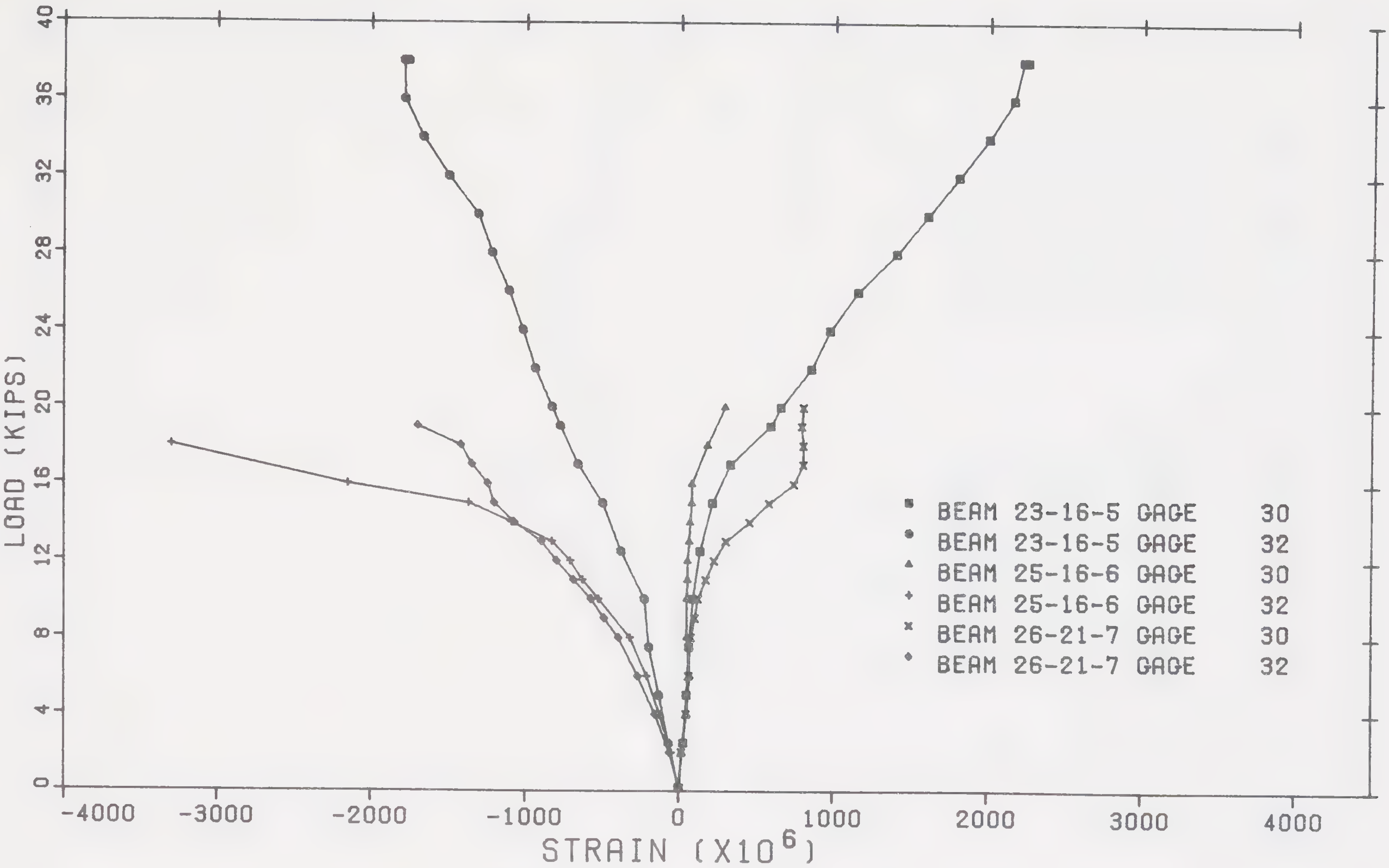


FIG. 4.94 LOAD VS. STRAIN FOR GAGES 29 TO 32

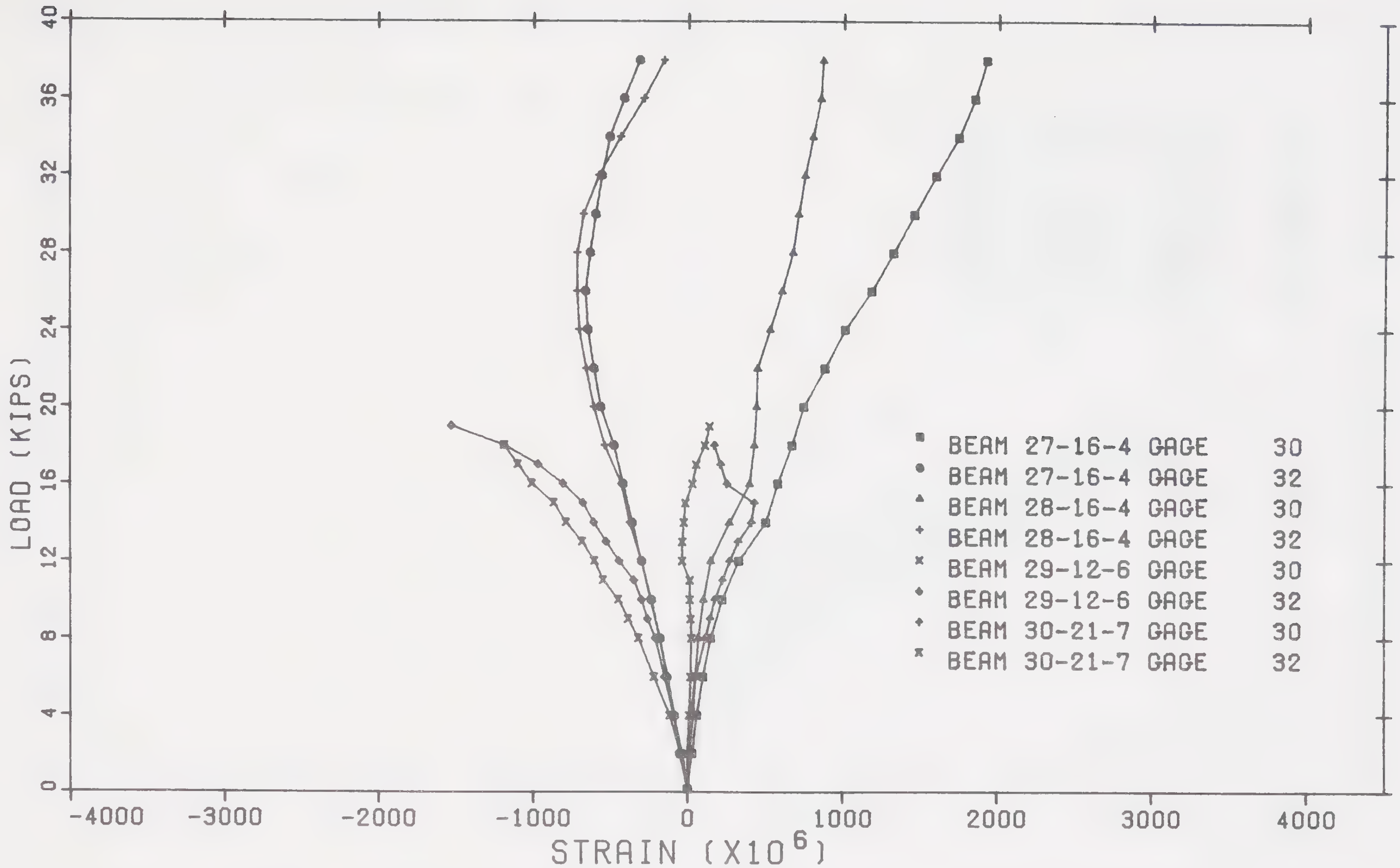


FIG. 4.95 LOAD VS. STRAIN FOR GAGES 29 TO 32

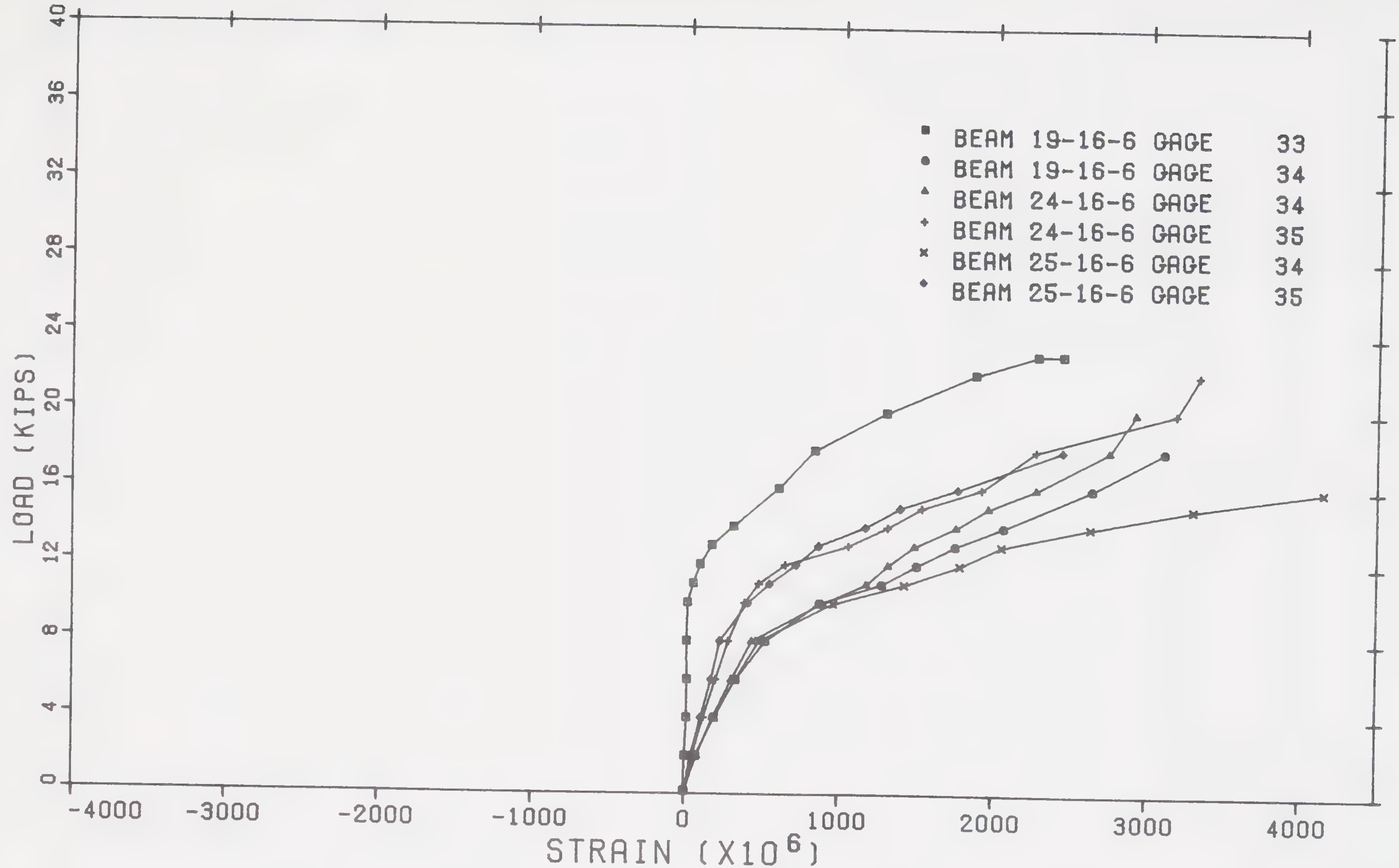


FIG. 4.96 LOAD VS. STRAIN FOR GAGES 33 TO 36

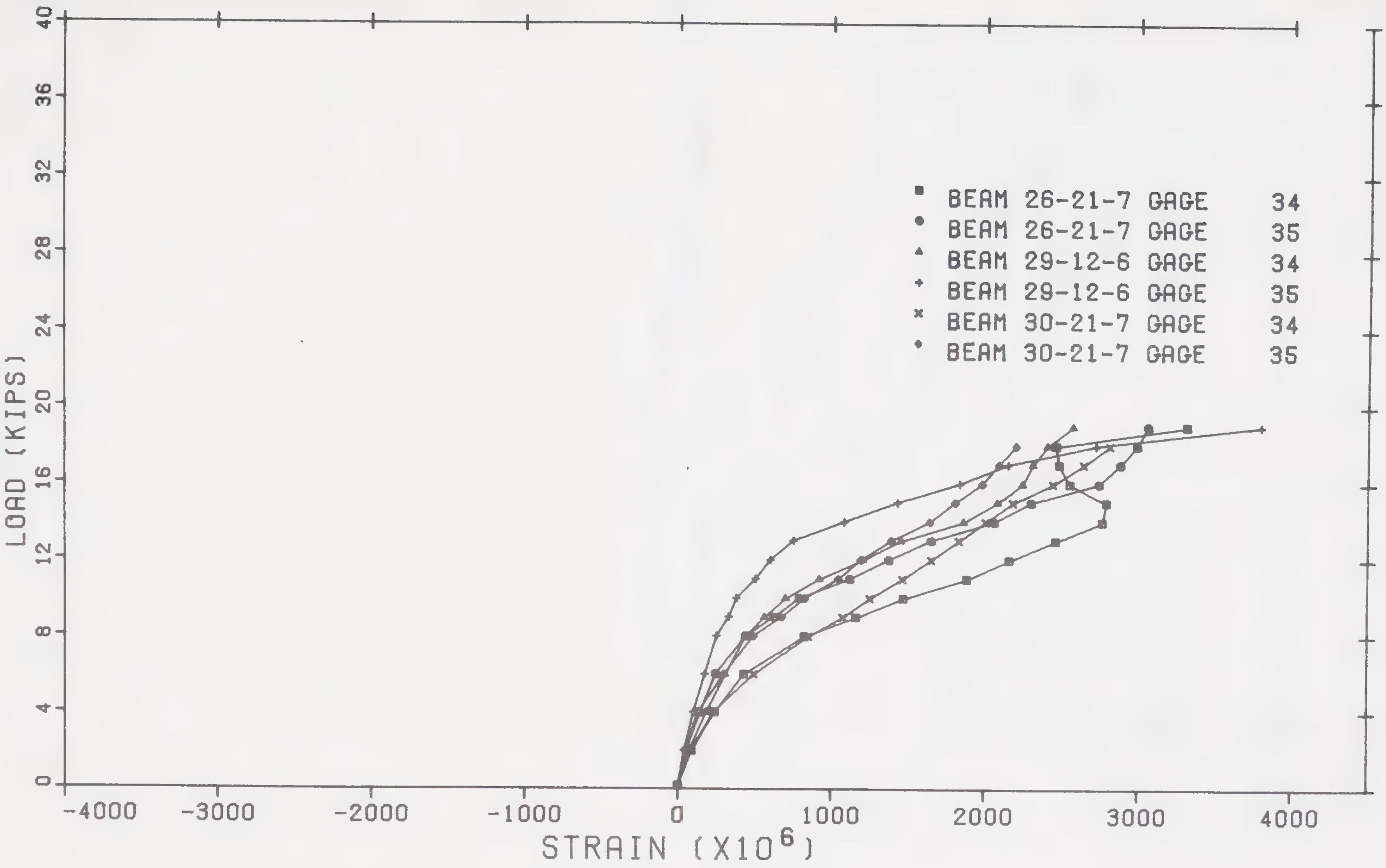


FIG. 4.97 LOAD VS. STRAIN FOR GAGES 33 TO 36

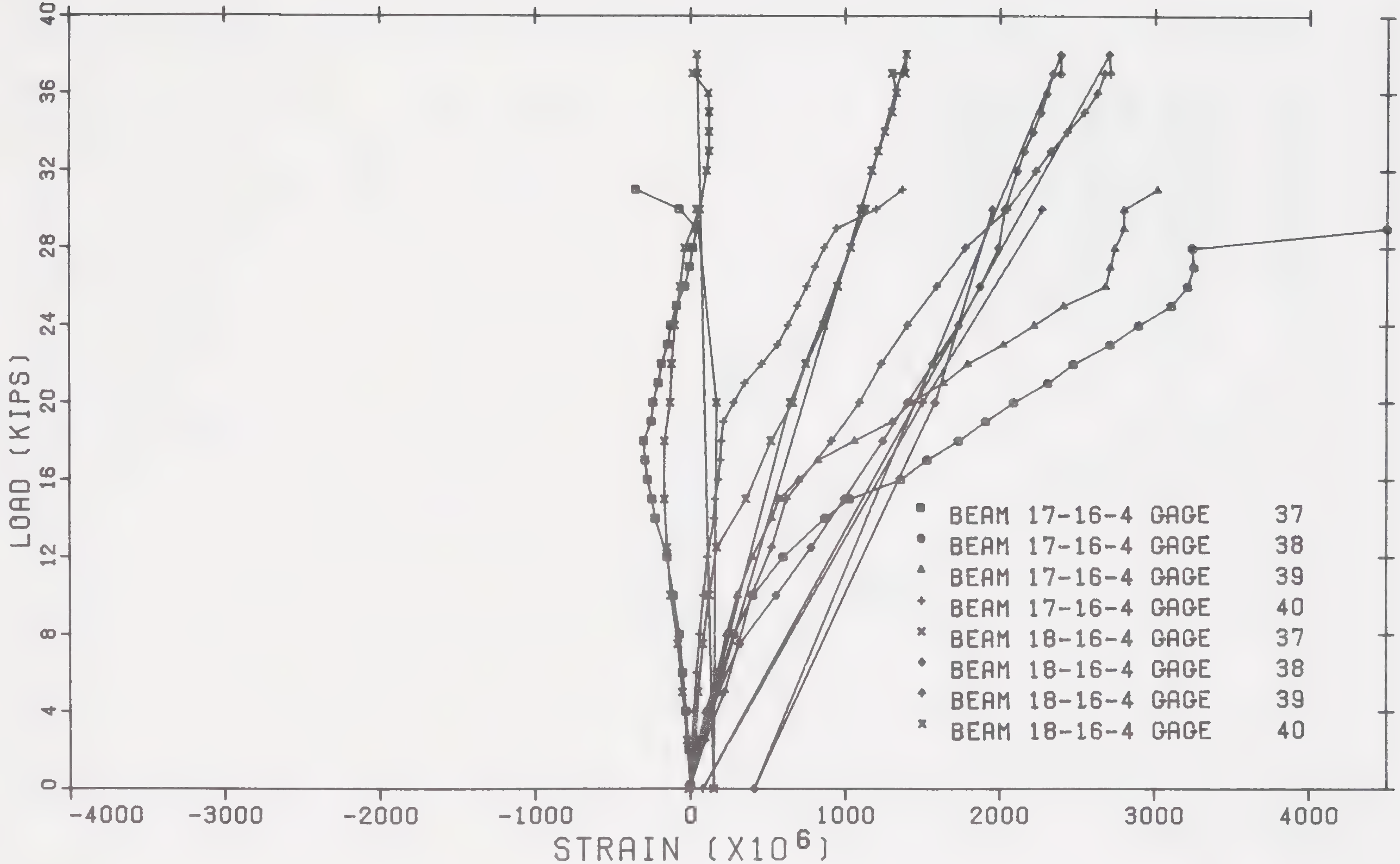


FIG. 4.98 LOAD VS. STRAIN FOR GAGES 37 TO 40

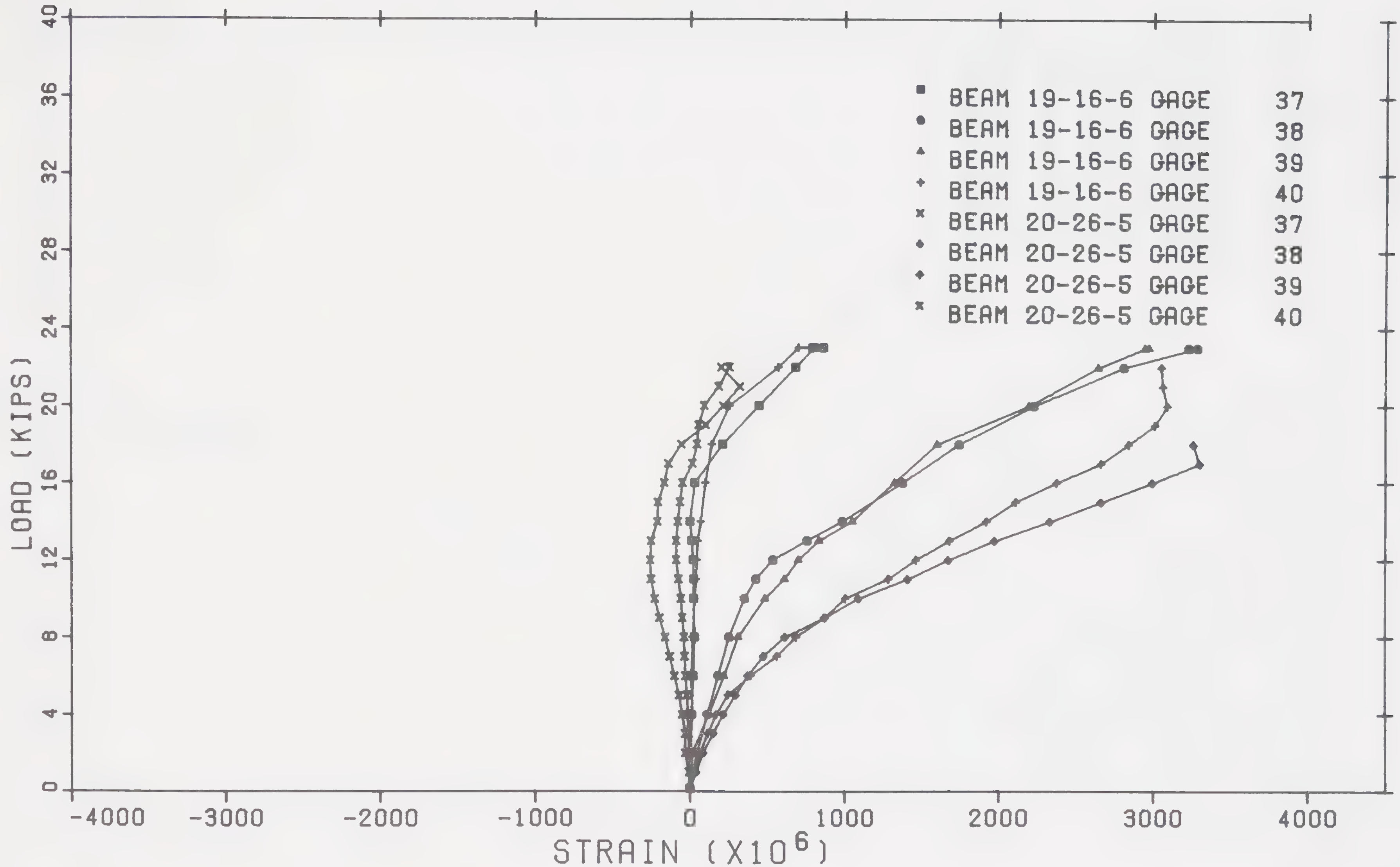


FIG. 4.99 LOAD VS. STRAIN FOR GAGES 37 TO 40

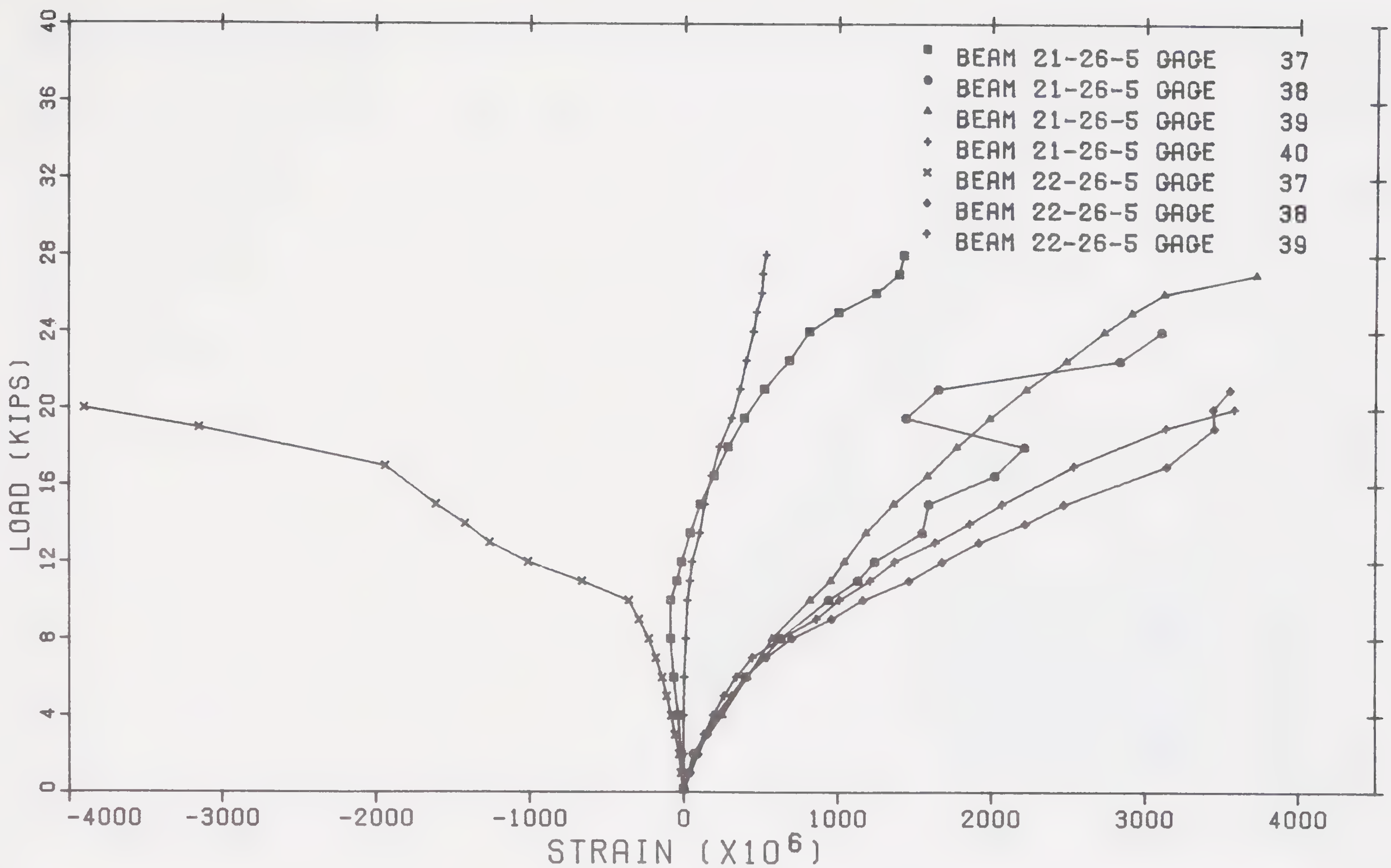


FIG. 4.100 LOAD VS. STRAIN FOR GAGES 37 TO 40

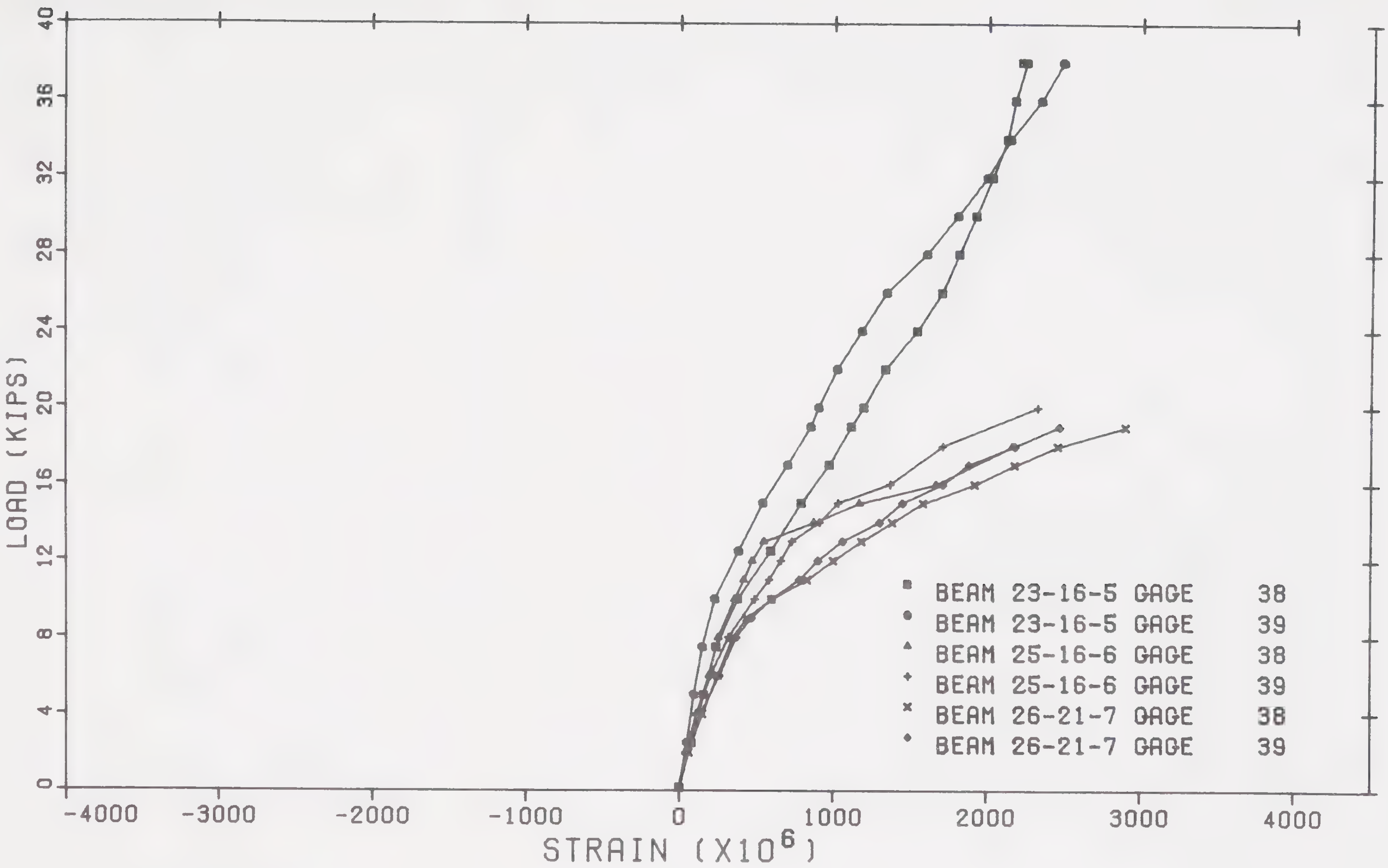


FIG. 4.101 LOAD VS. STRAIN FOR GAGES 37 TO 40

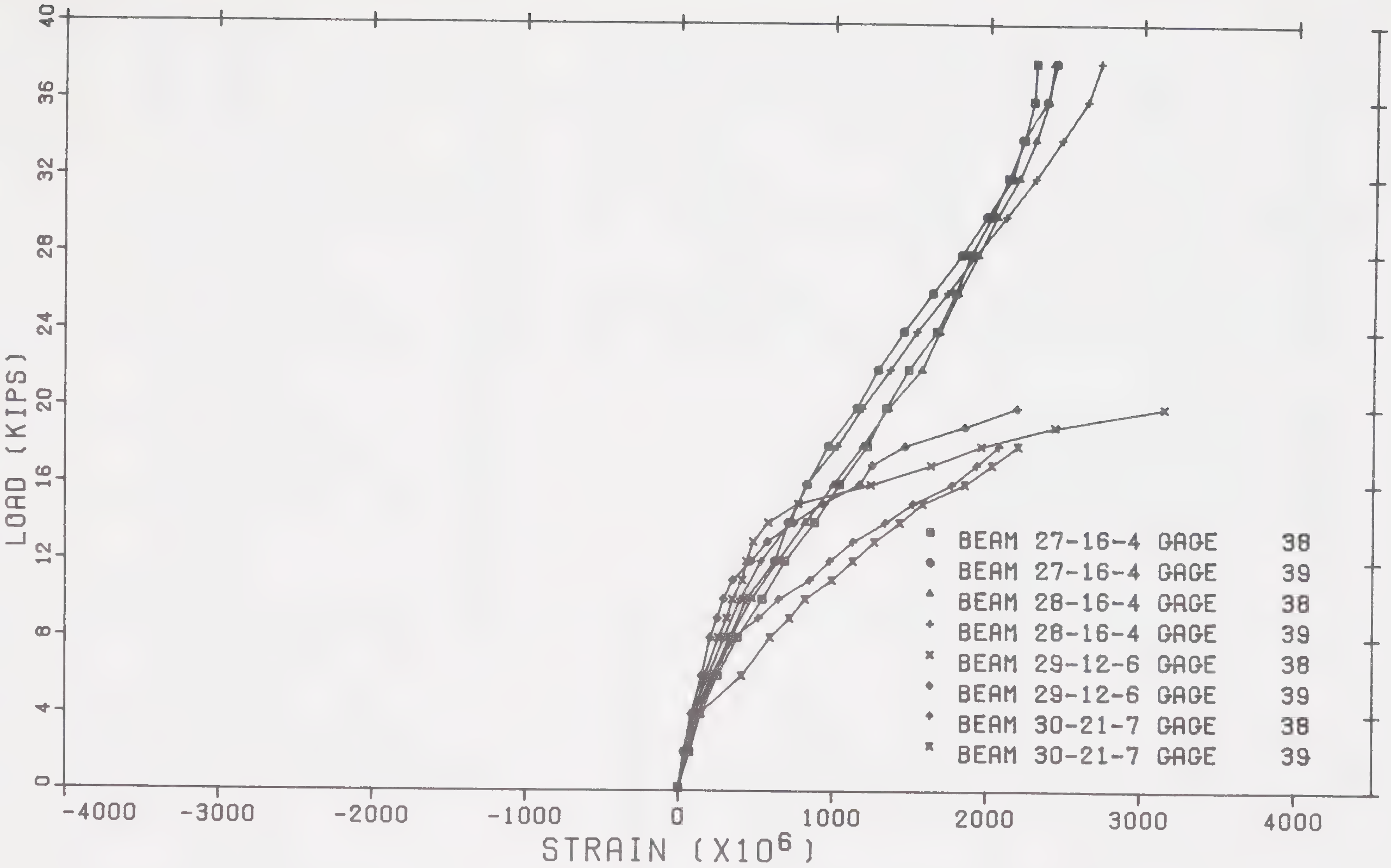


FIG. 4.102 LOAD VS. STRAIN FOR GAGES 37 TO 40

4.7 Moment Strain Relationships for the Concrete at the Beam Centerline

The concrete strains over the depth of the beam at mid-span are plotted for selected moments in Figures 4.103 to 4.108. The plotted strains were those obtained from the mechanical Demec strain gages over an 8 inch gage length and the moments were calculated at the beam centerline from the applied loads. The plots are in numerical order of the beam tests. The zero reading for each beam was taken just prior to the release of the prestressing force. The first plot for each beam is the strain distribution after transfer of the prestressing force to the beam. The second and subsequent plots are for strains measured during the test at applied moments 0.0 in-kips to the maximum moment for which readings were taken in 500 in-kips increments. The maximum moment for which strain data is available is shown in the final strain distribution for each beam.

4.8 Illustrative Cracking and Failure Patterns

Figures 4.109 to 4.138 include photographic plates of the cracking and failure patterns of the thirty beams tested. They contain close-up photographs of the beam in the region of the failure and full-length photographs of the beam at failure. The cracks were traced on the beam surface and numbered with the corresponding load increments. The loads and increments are tabulated in Appendix B.

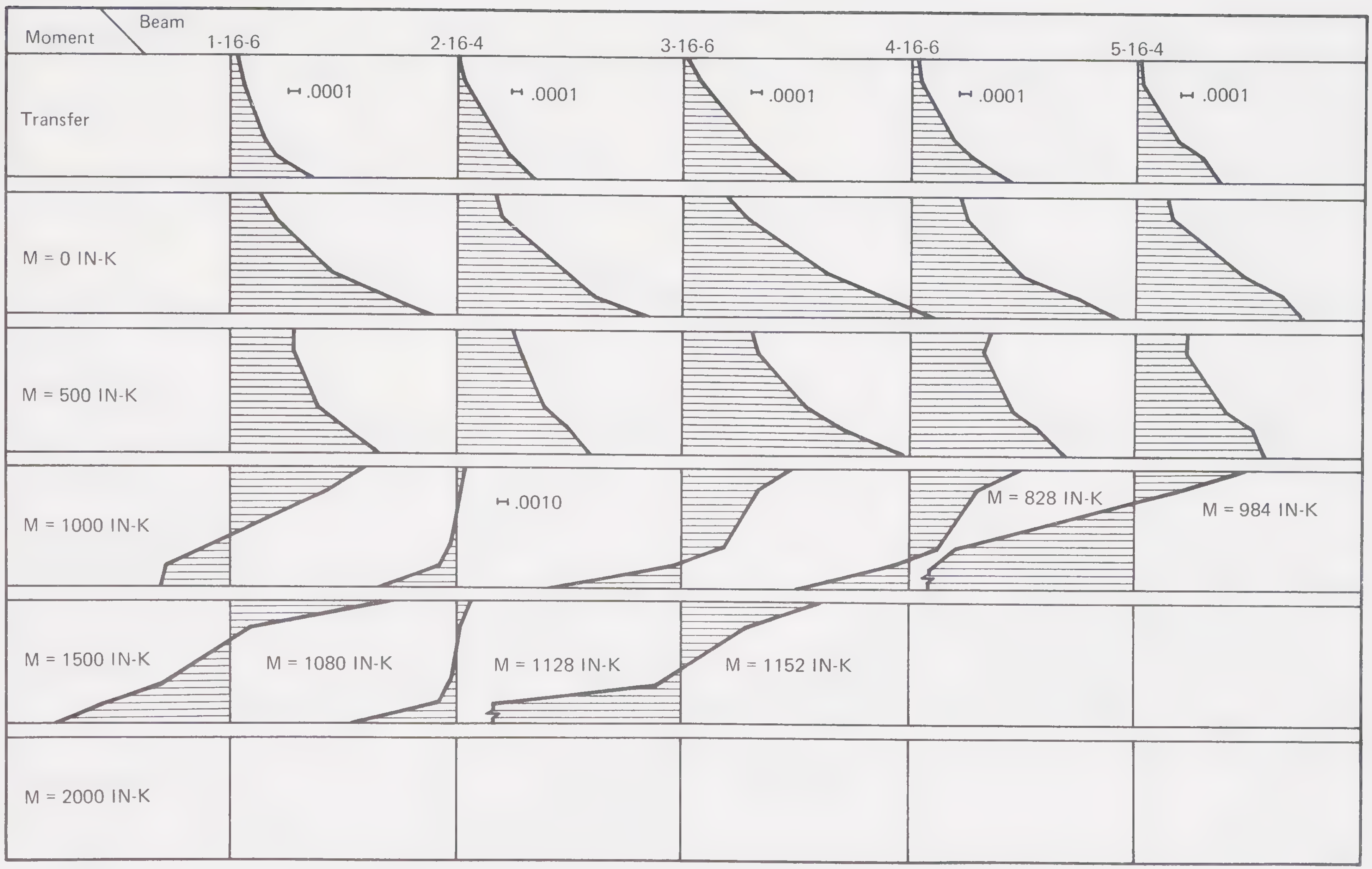


FIGURE 4.103. Concrete Strain Distribution at the Beam Centerline

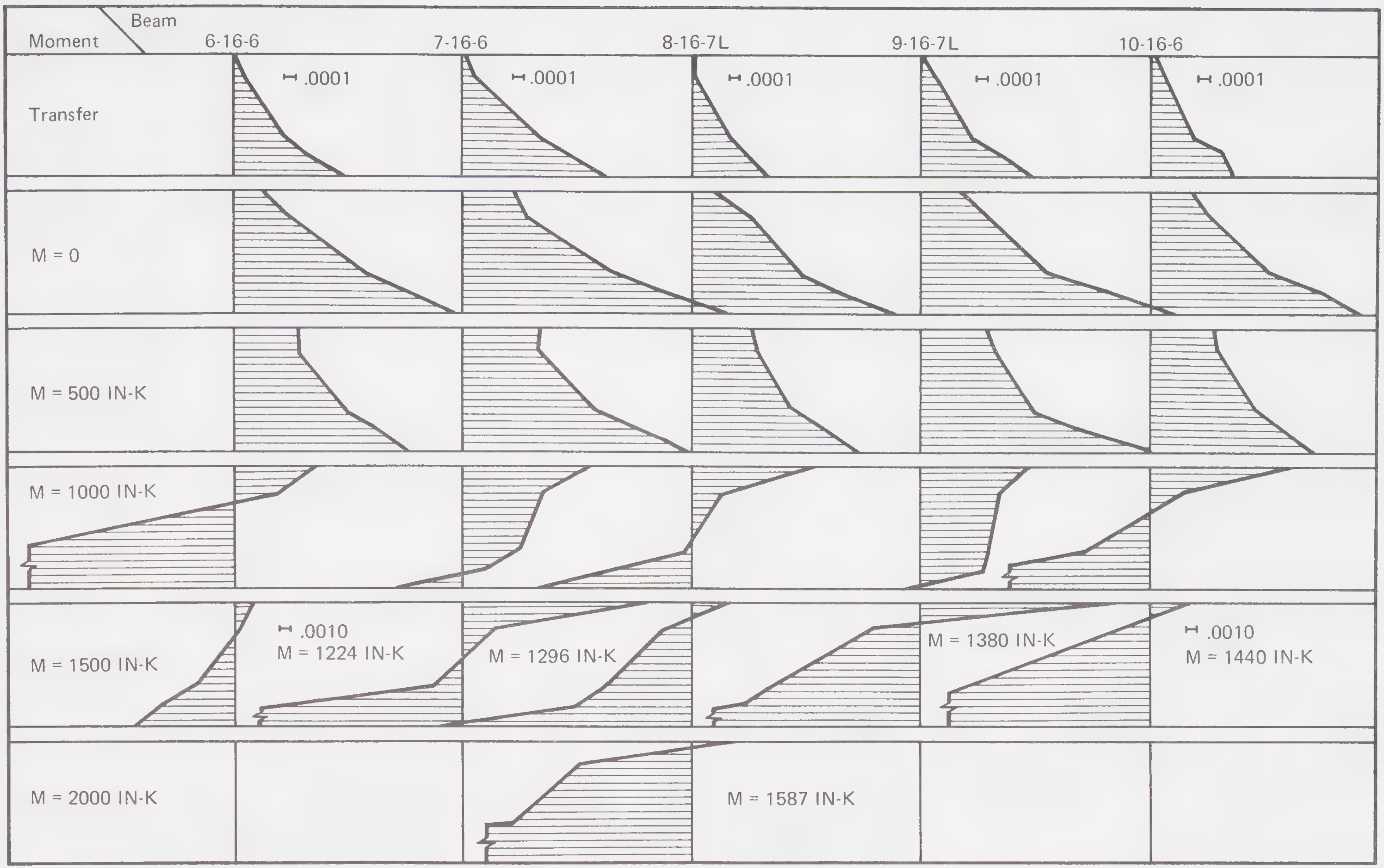


FIGURE 4.104. Concrete Strain Distribution at the Beam Centerline

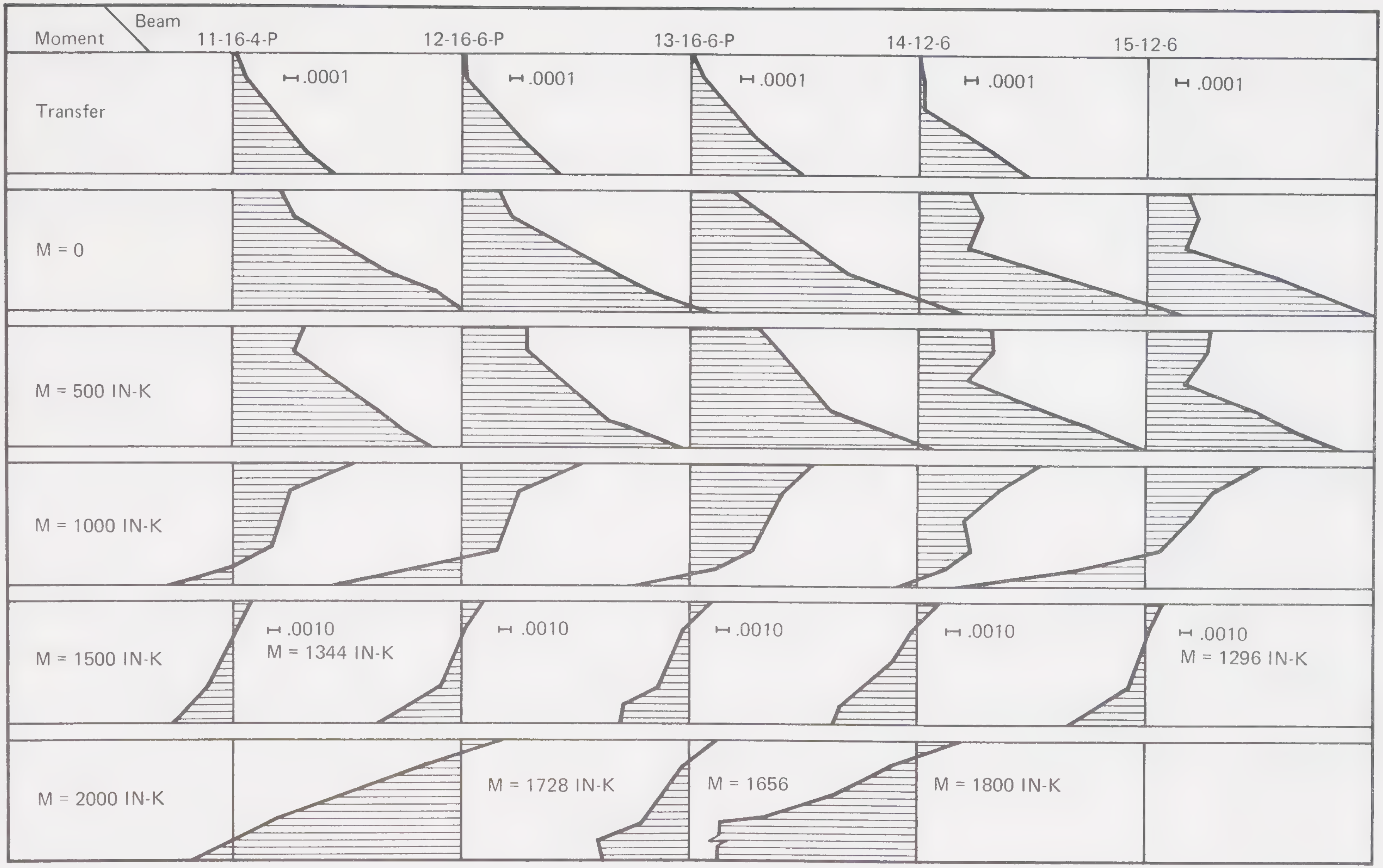


FIGURE 4.105. Concrete Strain Distribution at the Beam Centerline

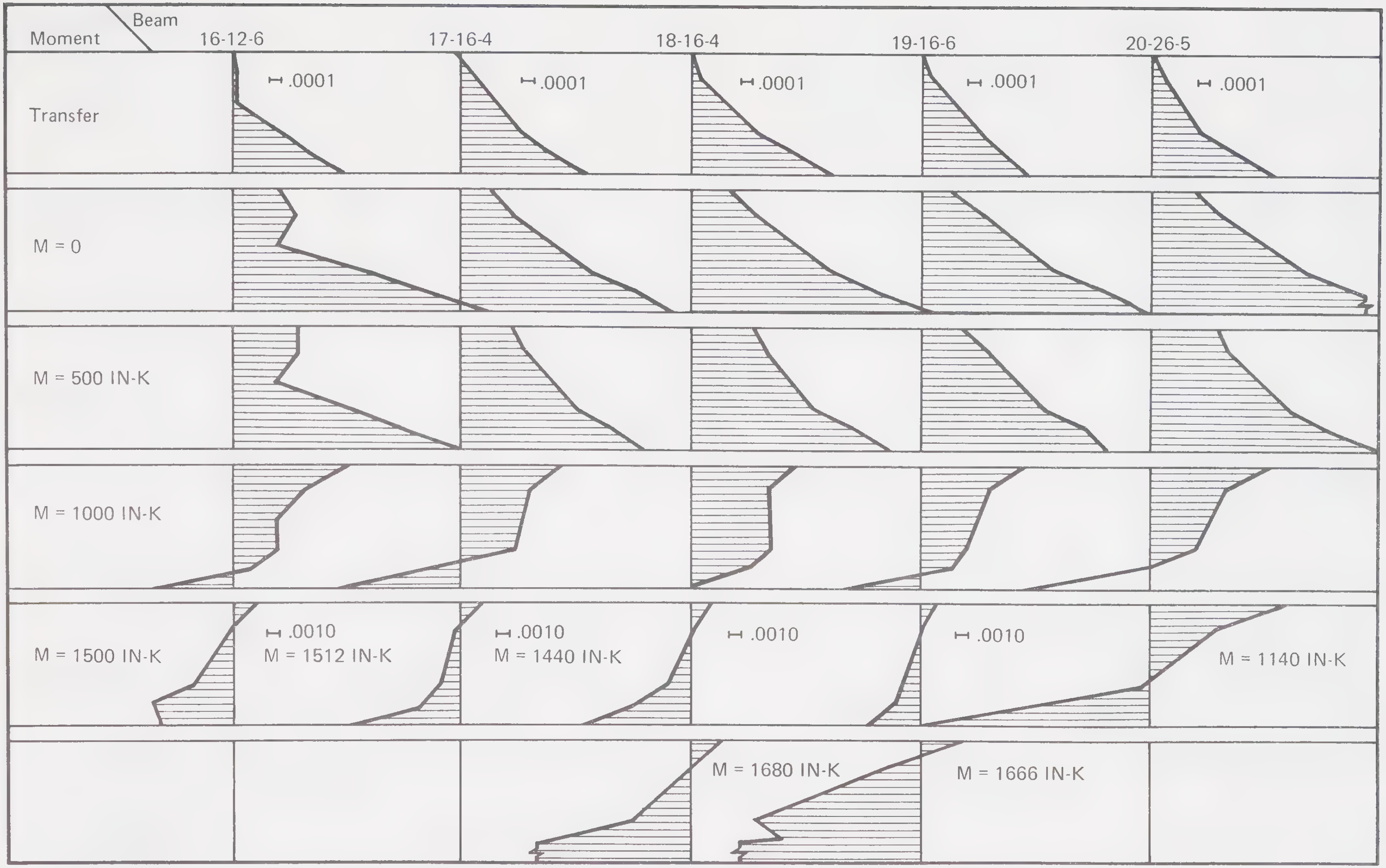


FIGURE 4.106. Concrete Strain Distribution at the Beam Centerline

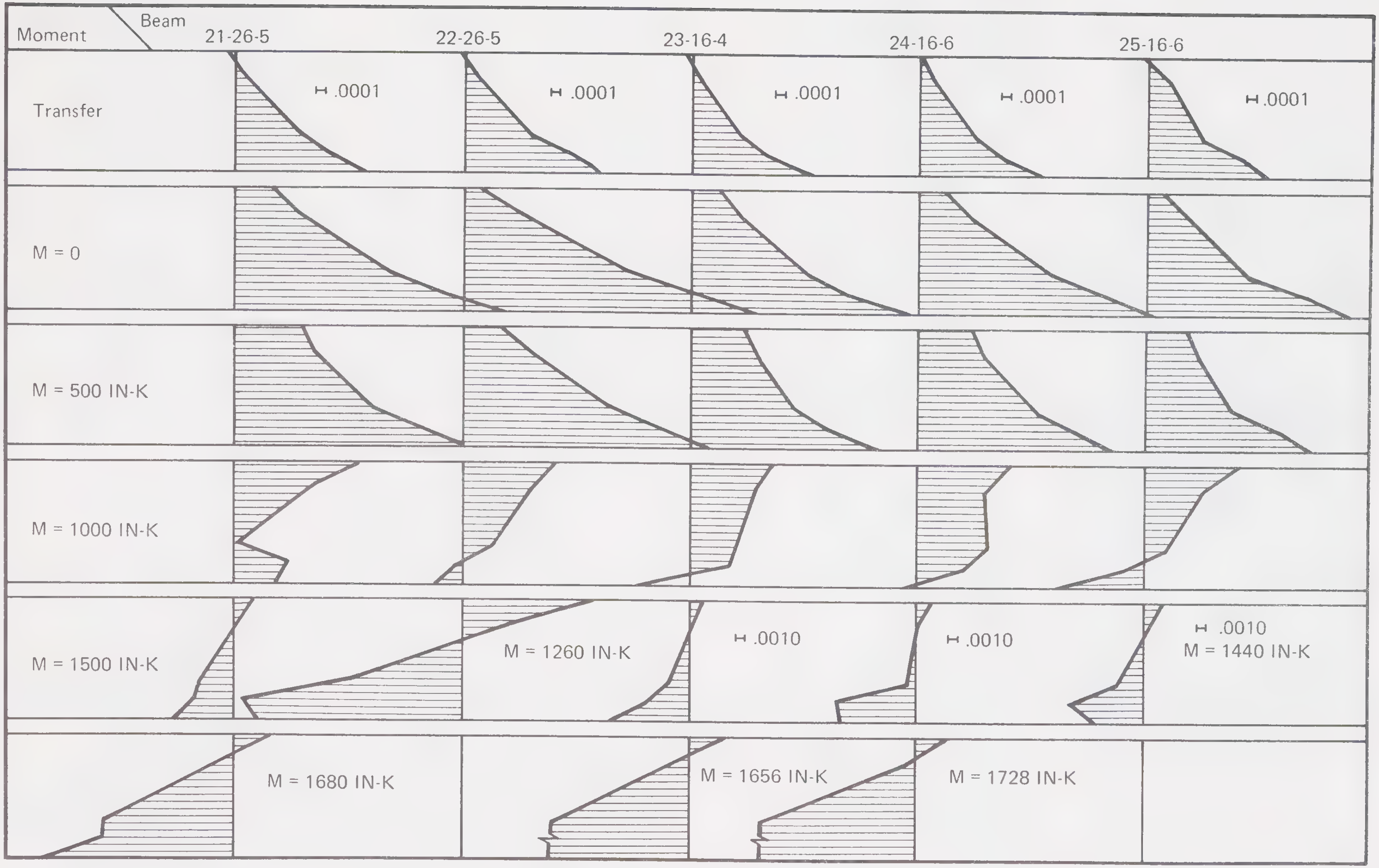


FIGURE 4.107. Concrete Strain Distribution at the Beam Centerline

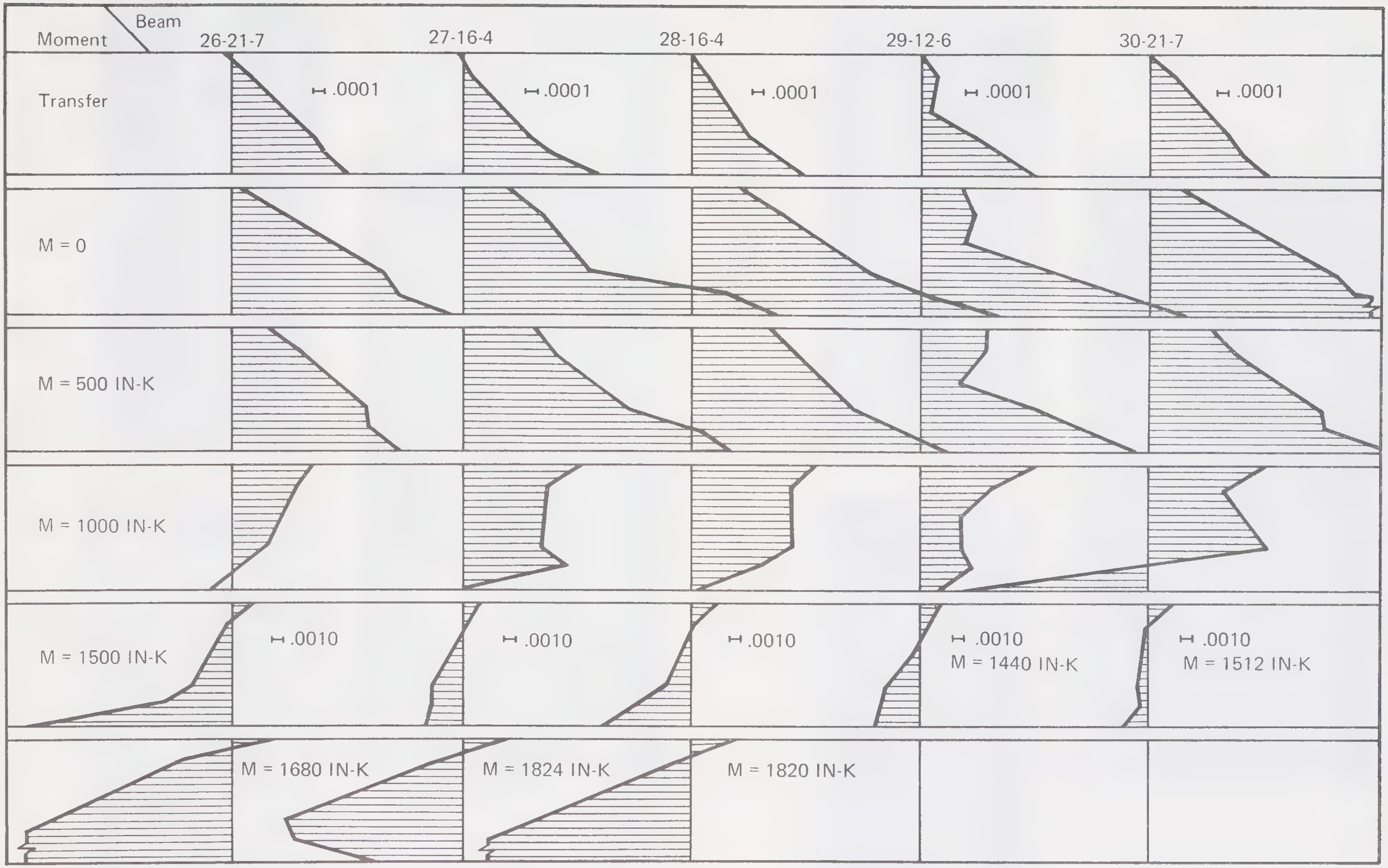


FIGURE 4.108. Concrete Strain Distribution at the Beam Centerline

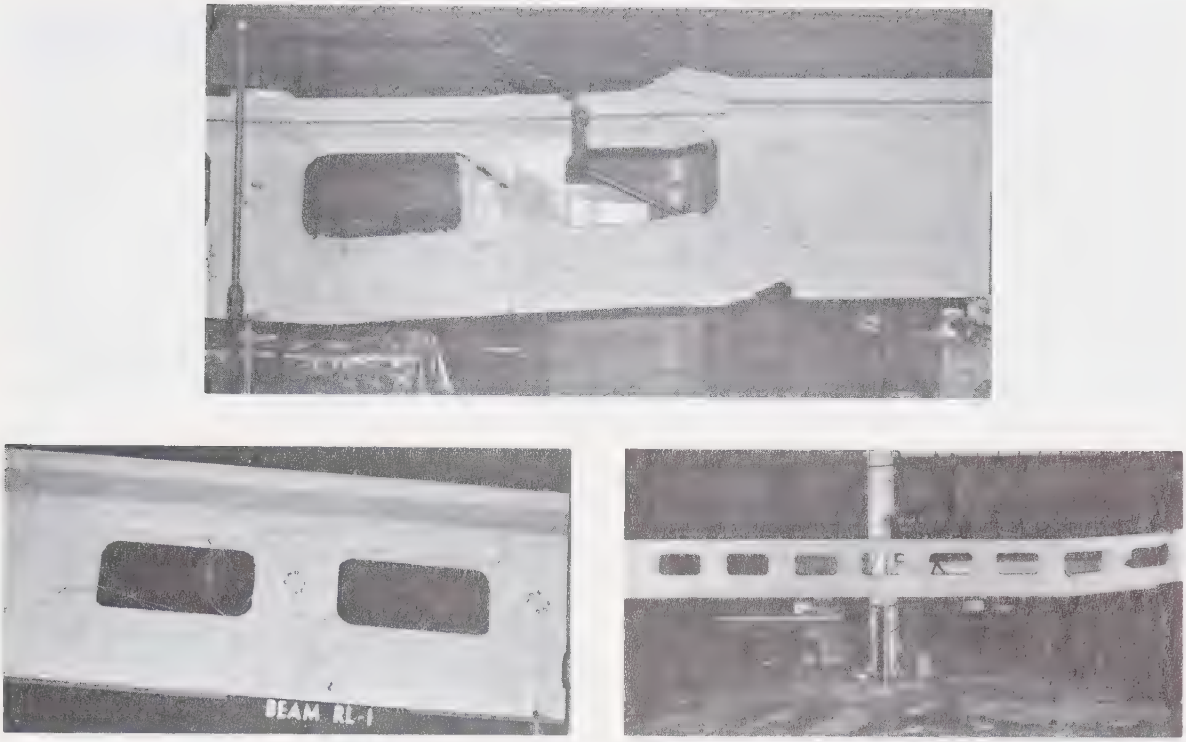


FIGURE 4.109 CRACKING AND FAILURE PATTERN OF BEAM 1-16-6

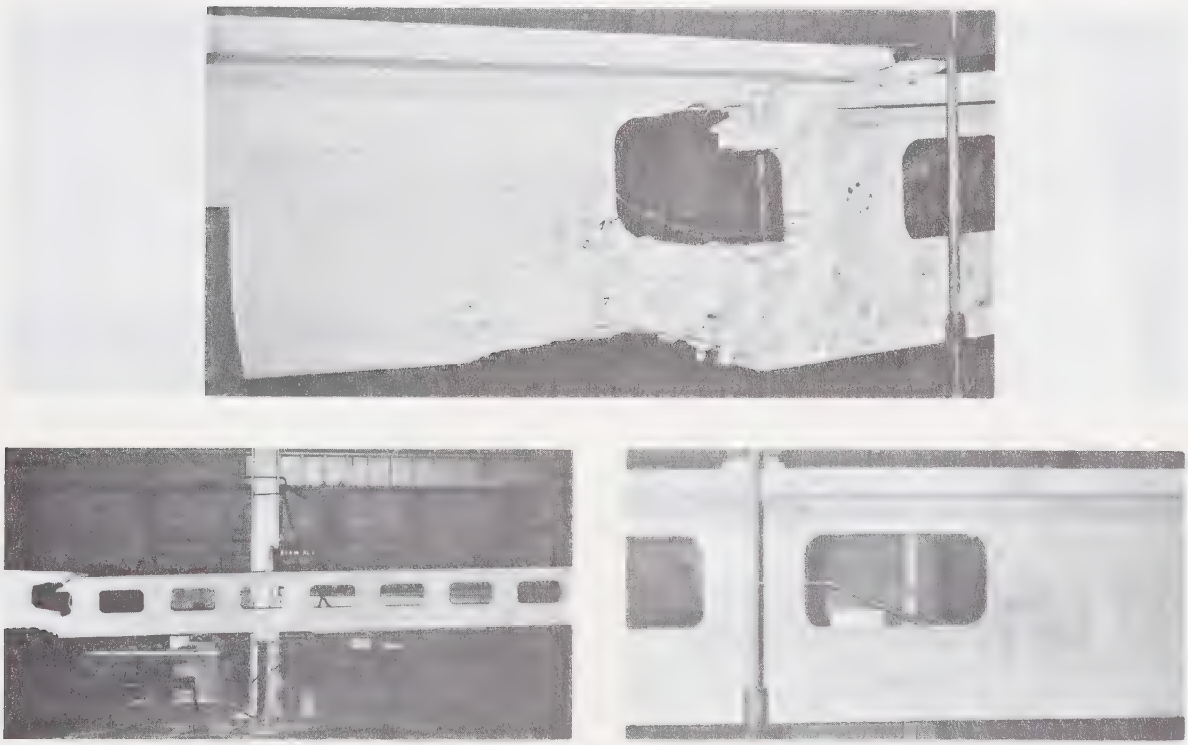


FIGURE 4.110 CRACKING AND FAILURE PATTERN OF BEAM 2-16-4

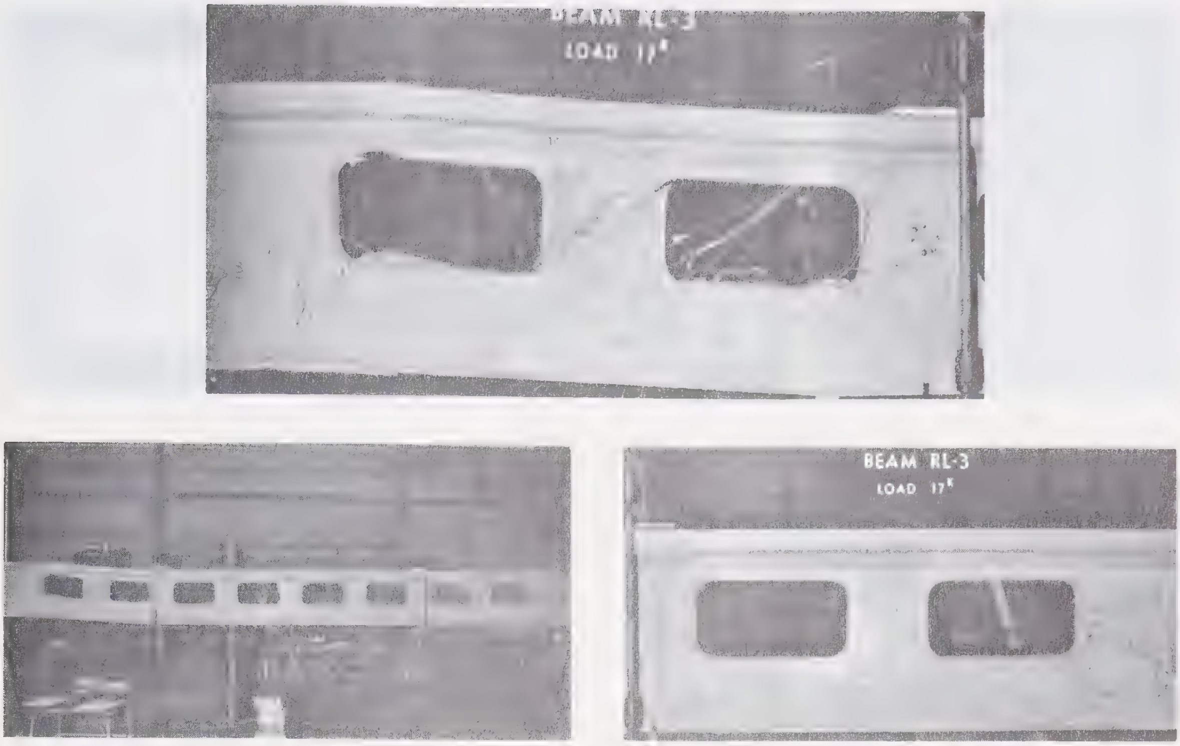


FIGURE 4.111 CRACKING AND FAILURE PATTERN OF BEAM 3-16-6

FIGURE 4.112 CRACKING AND FAILURE PATTERN OF BEAM 4-16-6

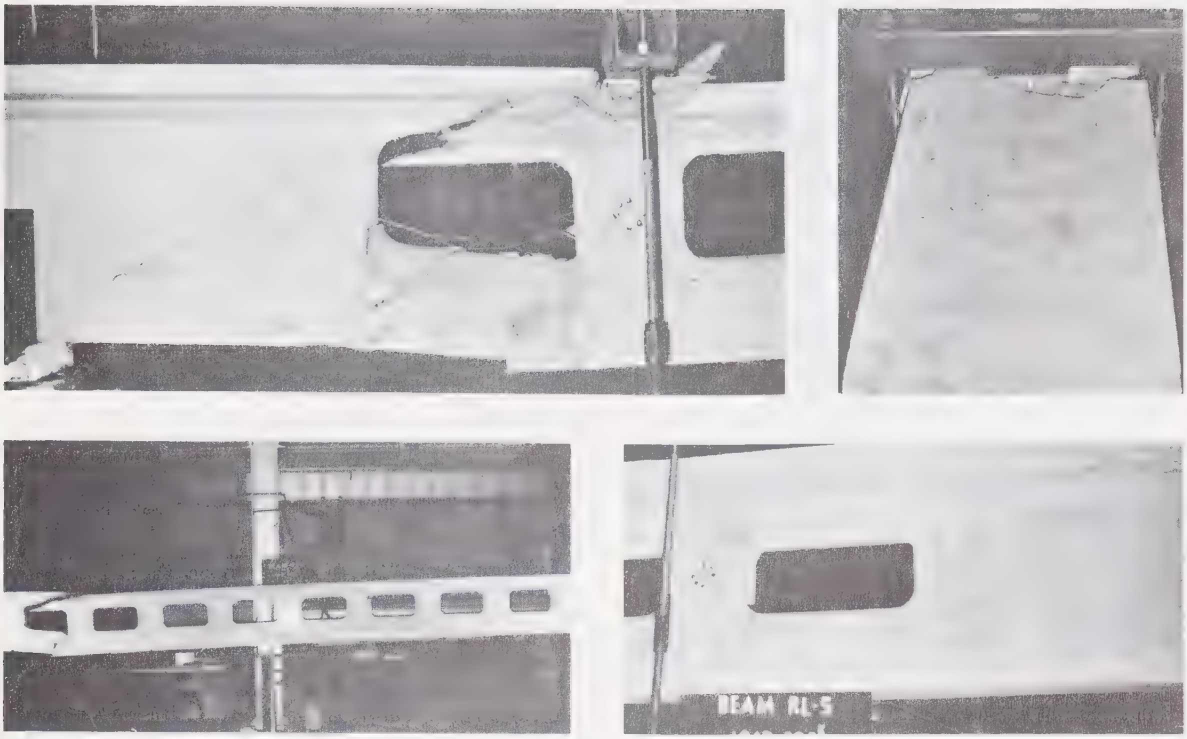


FIGURE 4.113 CRACKING AND FAILURE PATTERN OF BEAM 5-16-4

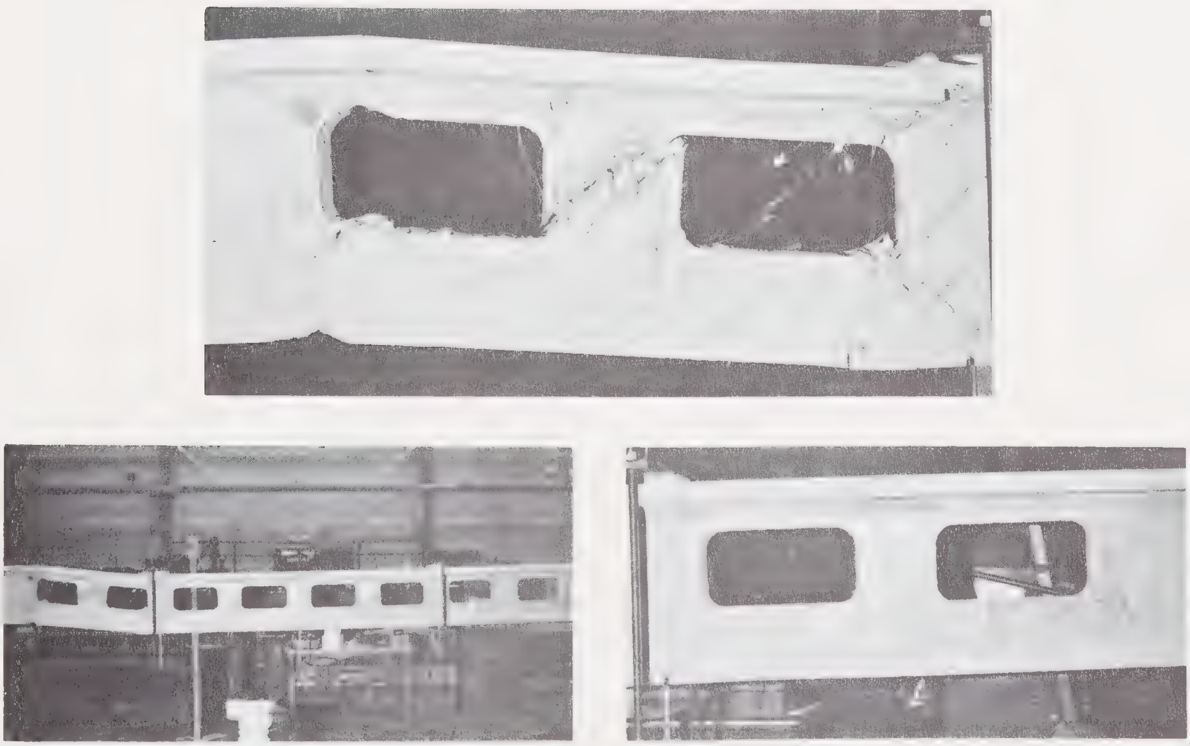


FIGURE 4.114 CRACKING AND FAILURE PATTERN OF BEAM 6-16-6

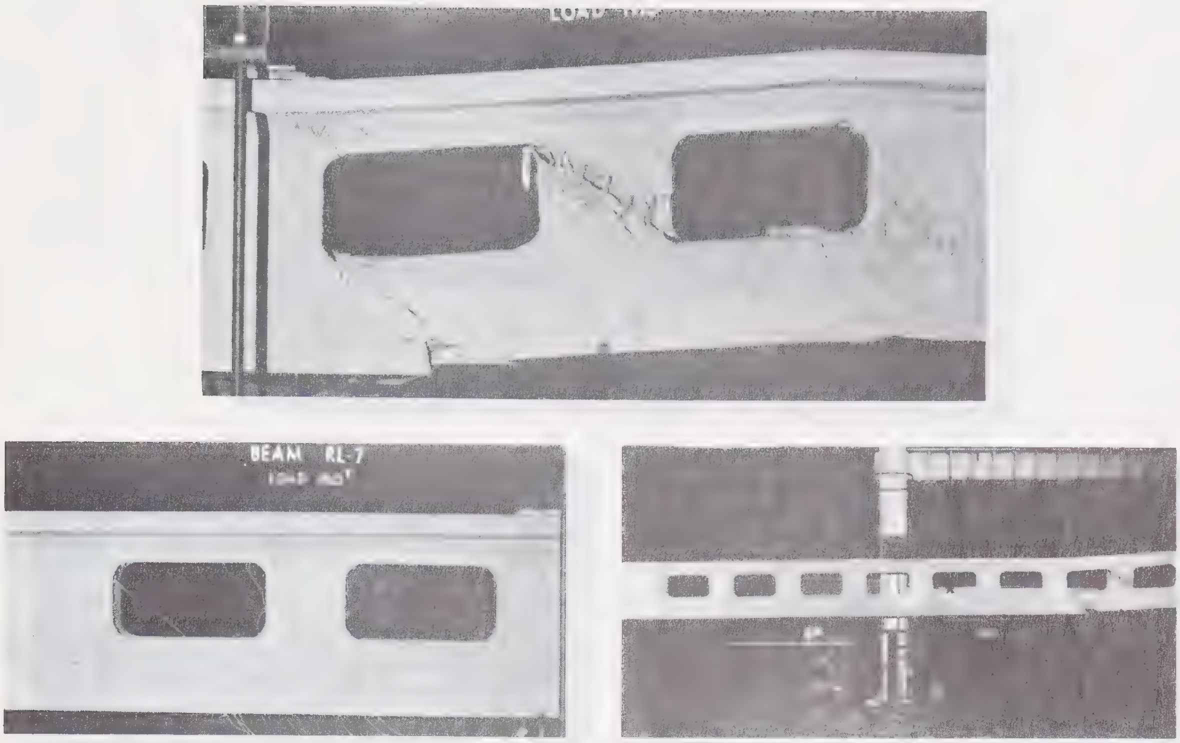


FIGURE 4.115 CRACKING AND FAILURE PATTERN OF BEAM 7-16-6

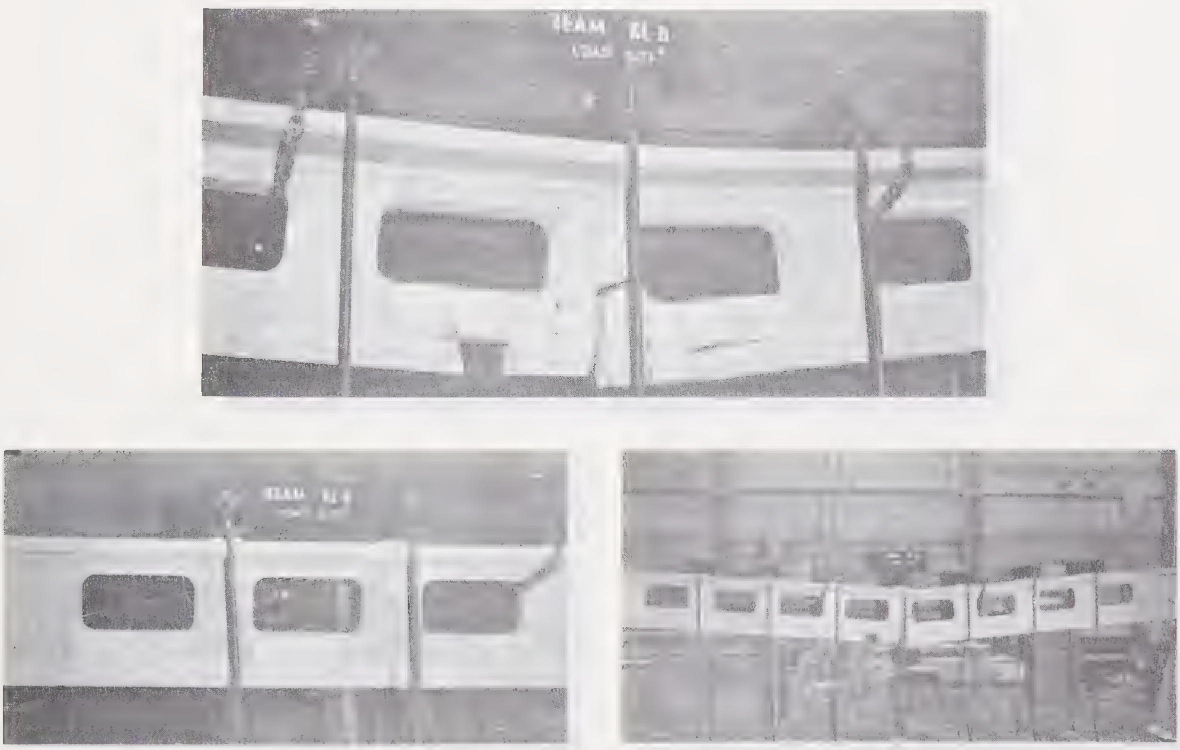


FIGURE 4.116 CRACKING AND FAILURE PATTERN OF BEAM 8-16-7L

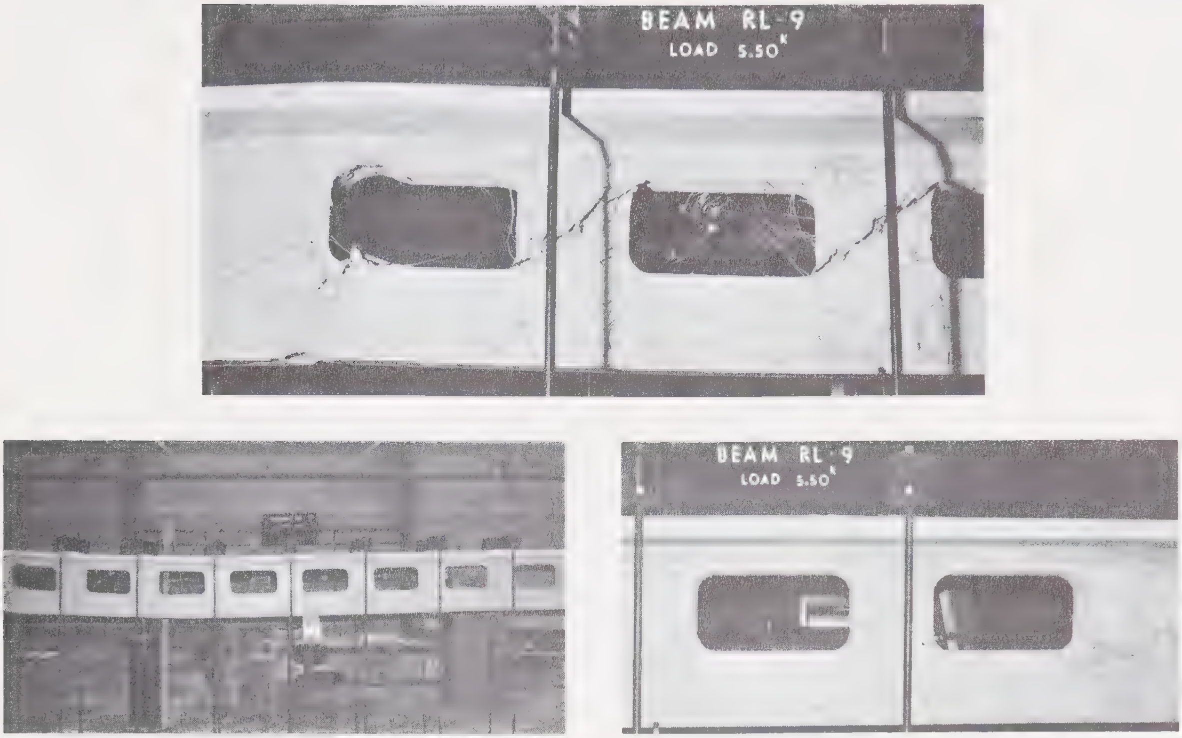


FIGURE 4.117 CRACKING AND FAILURE PATTERN OF BEAM 9-16-7L

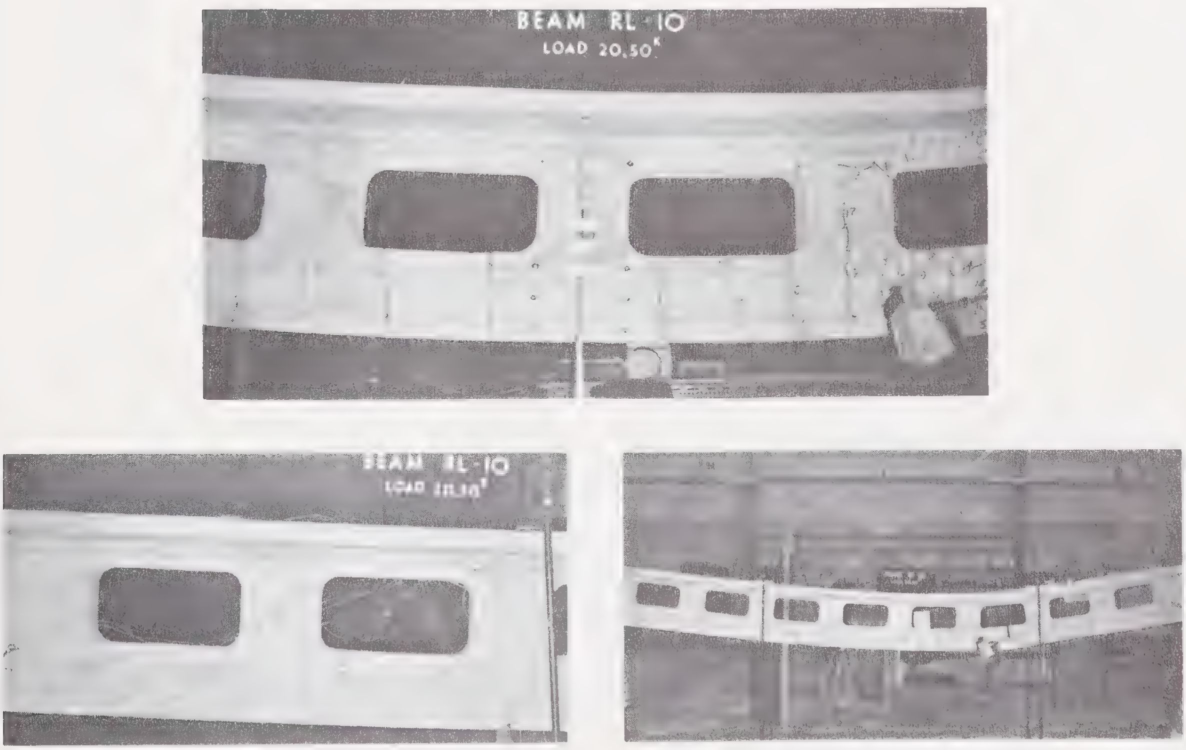


FIGURE 4.118 CRACKING AND FAILURE PATTERN OF BEAM 10-16-6

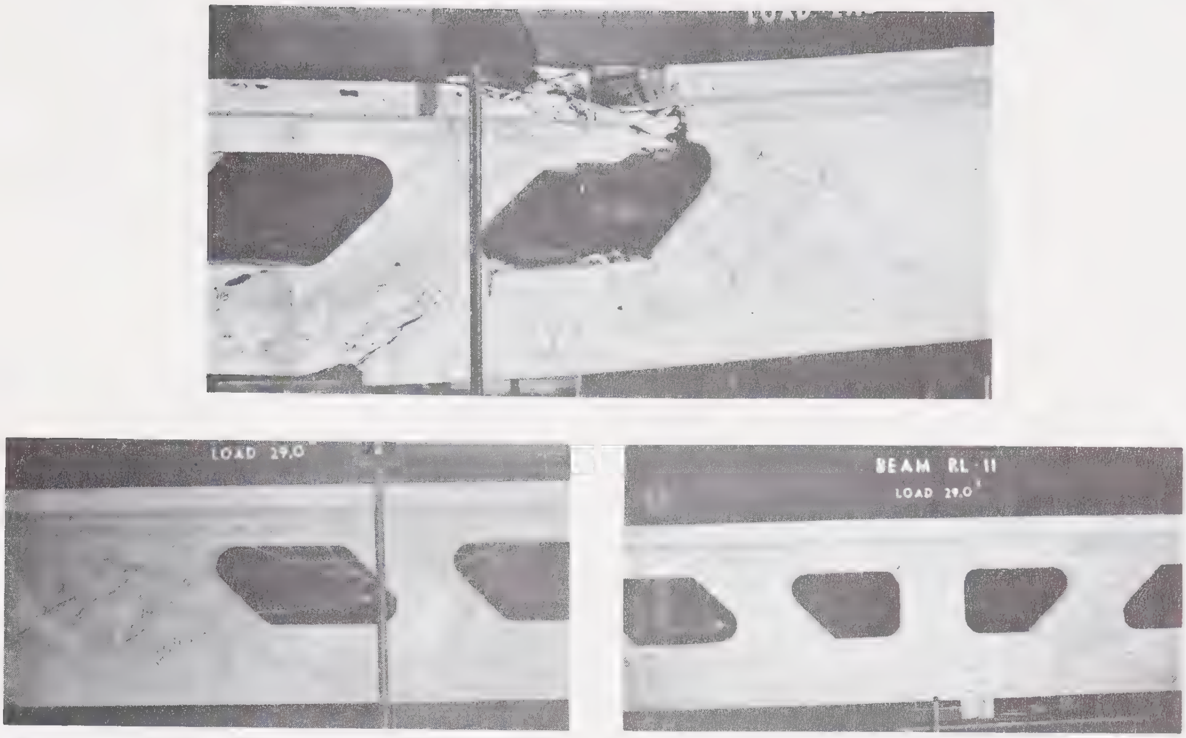


FIGURE 4.119 CRACKING AND FAILURE PATTERN OF BEAM 11-16-4-P

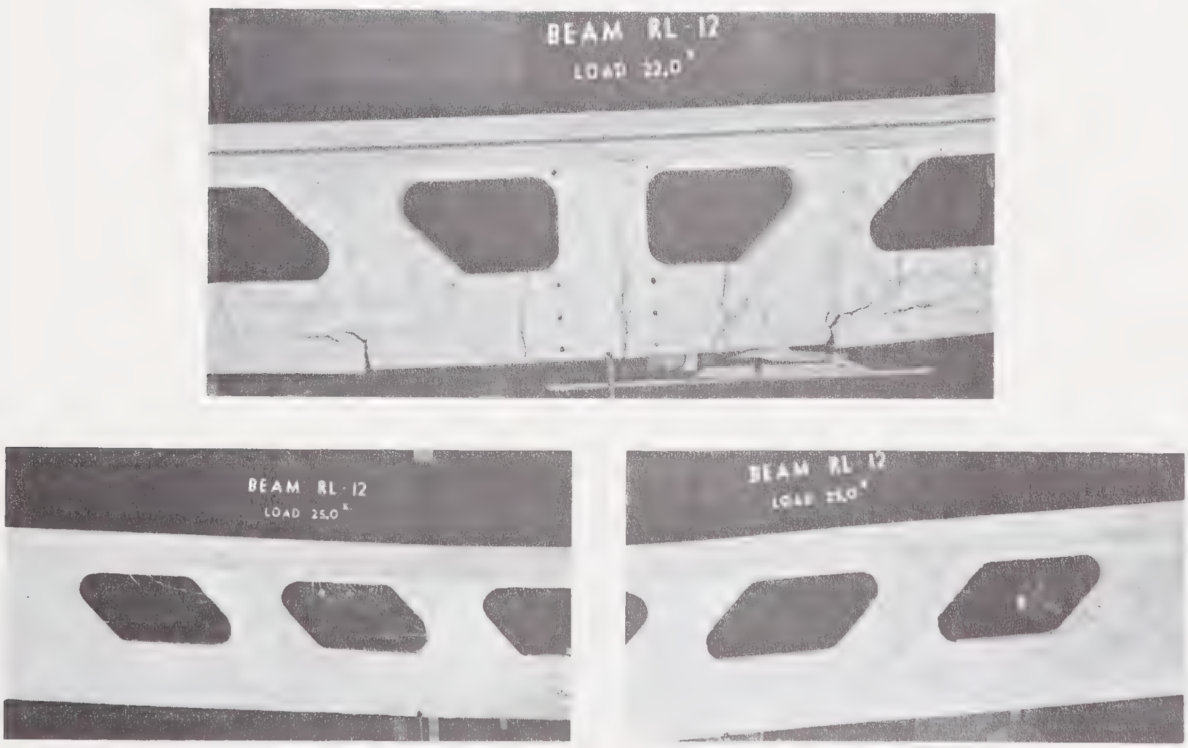


FIGURE 4.120 CRACKING AND FAILURE PATTERN OF BEAM 12-16-6-P

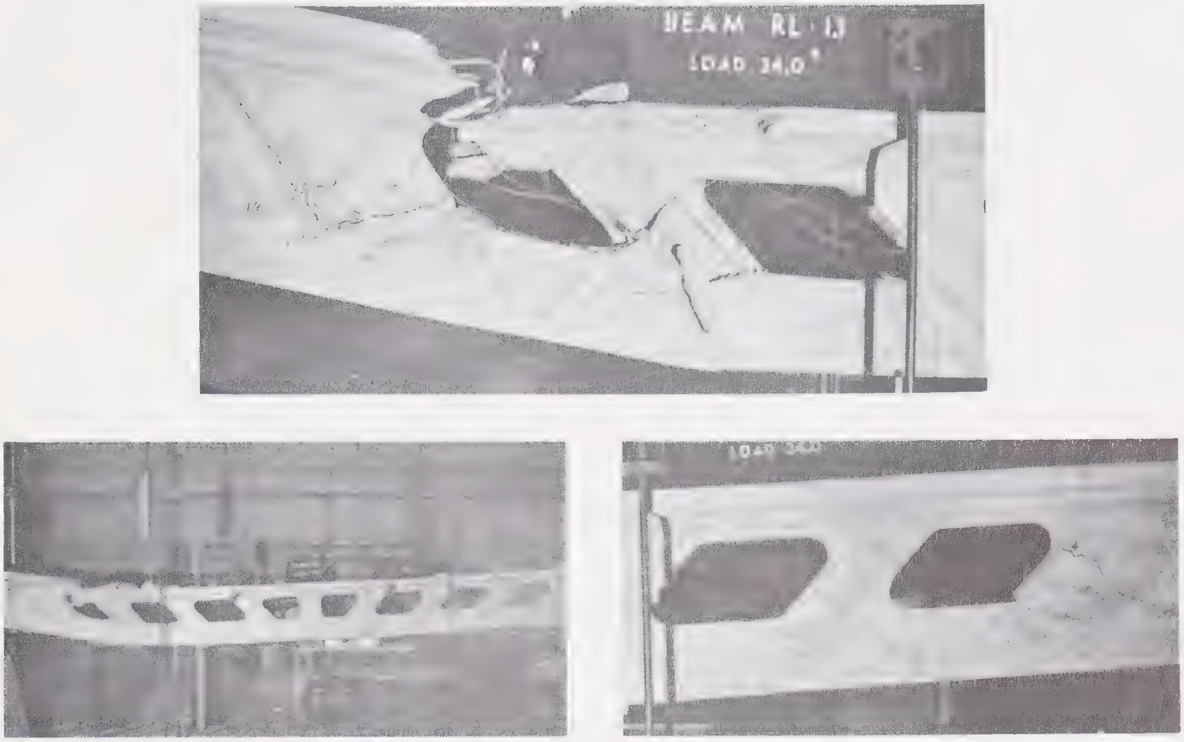


FIGURE 4.121 CRACKING AND FAILURE PATTERN OF BEAM 13-16-6-P

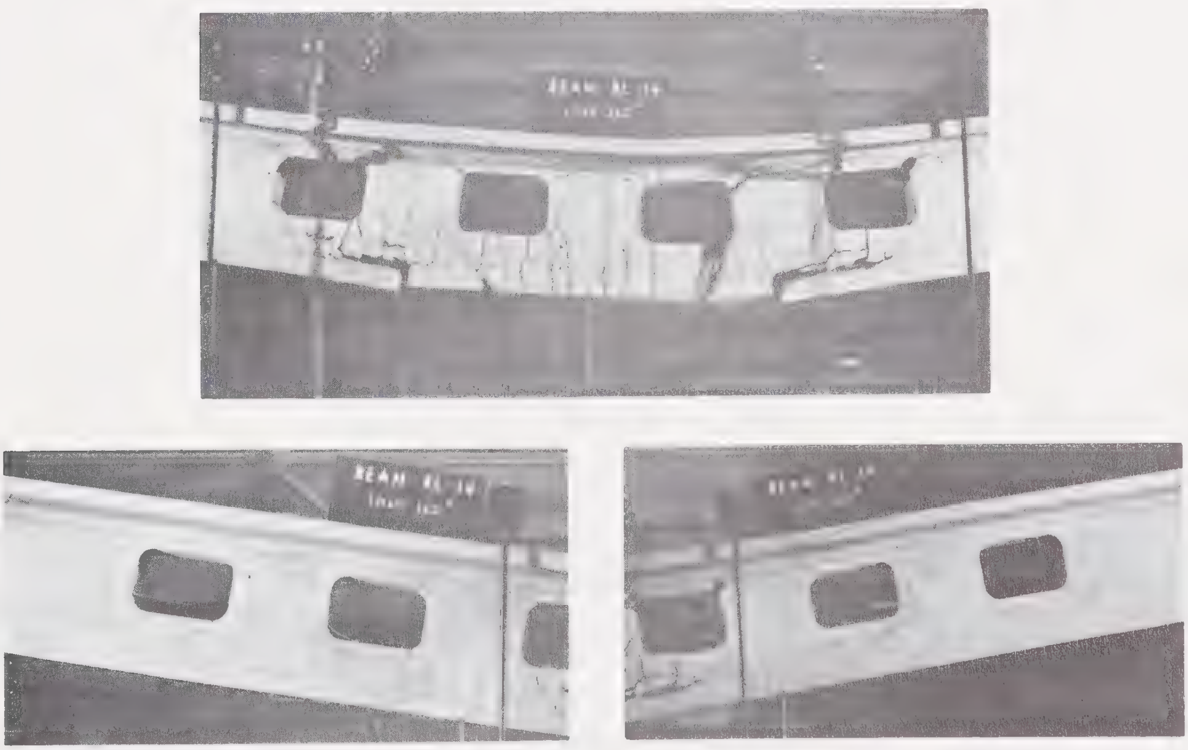


FIGURE 4.122 CRACKING AND FAILURE PATTERN OF BEAM 14-12-6

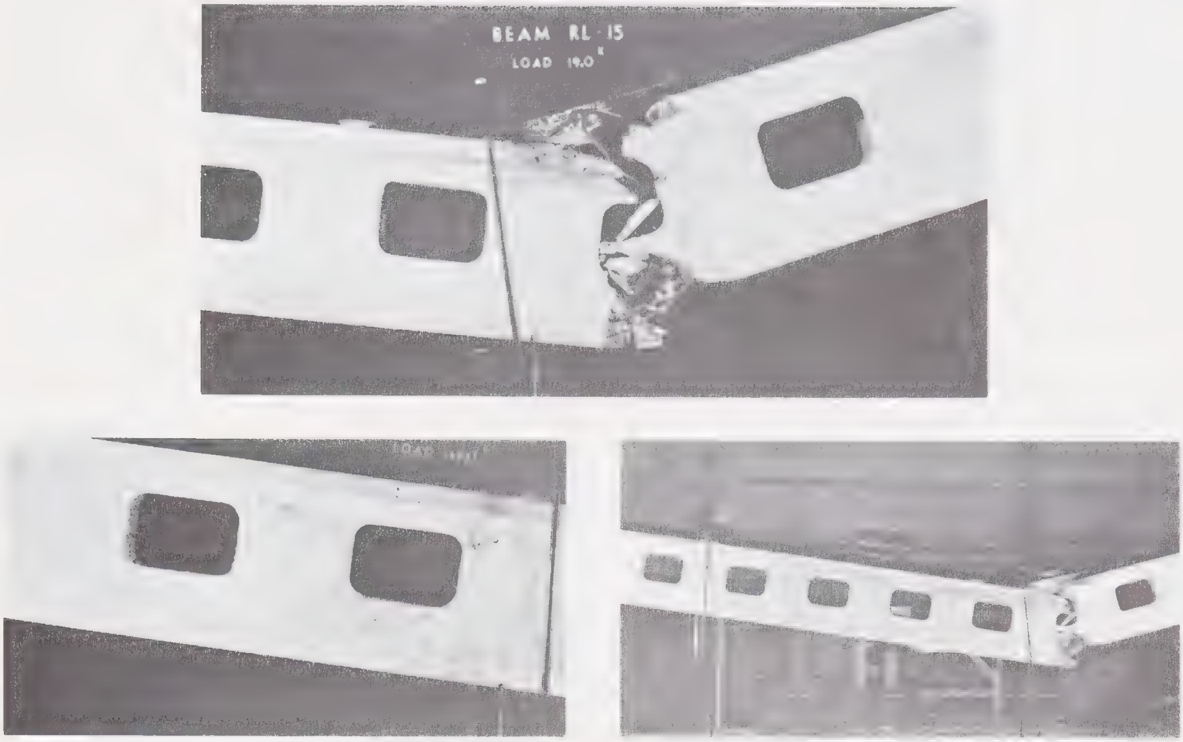


FIGURE 4.123 CRACKING AND FAILURE PATTERN OF BEAM 15-12-6

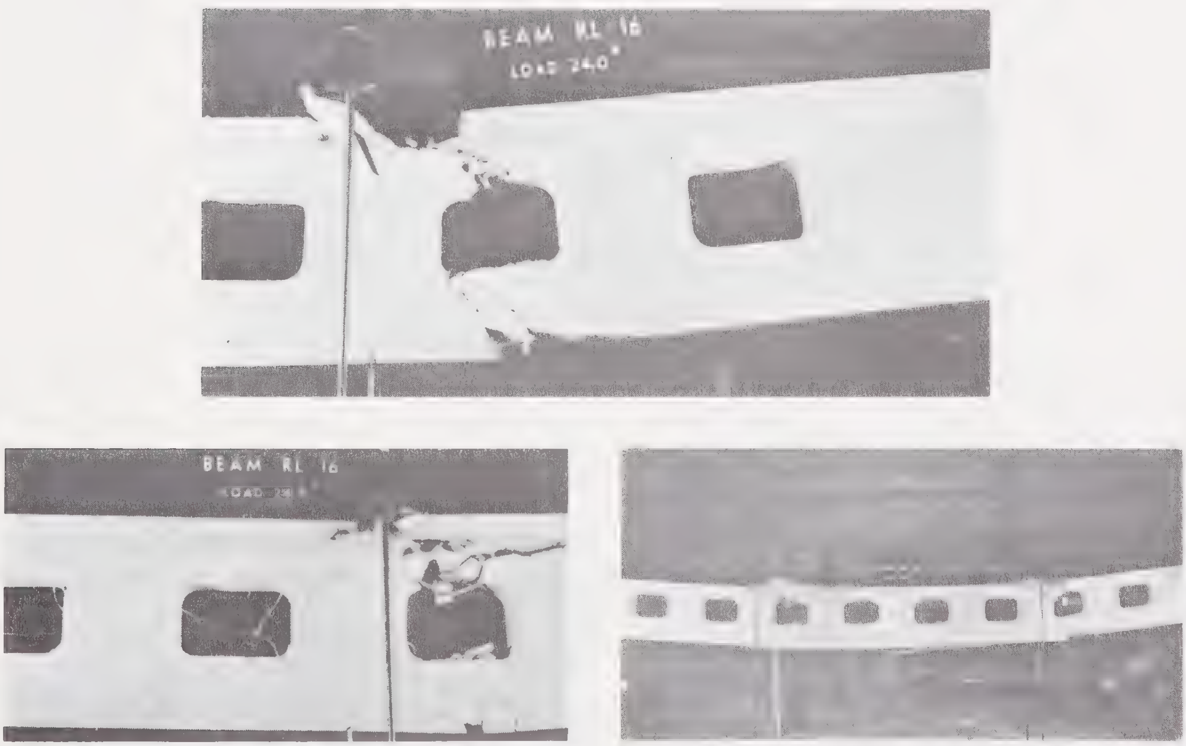


FIGURE 4.124 CRACKING AND FAILURE PATTERN OF BEAM 16-12-6

FIGURE 4.125 CRACKING AND FAILURE PATTERN OF BEAM 17-16-4

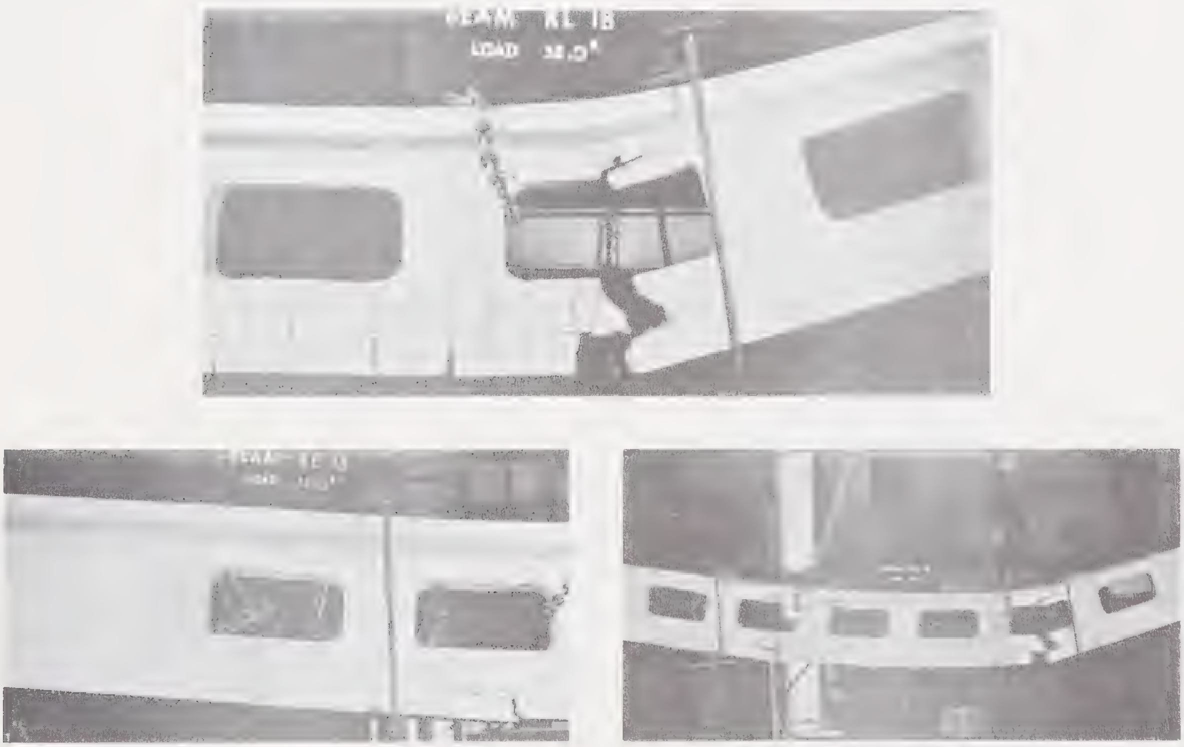


FIGURE 4.126 CRACKING AND FAILURE PATTERN OF BEAM 18-16-4

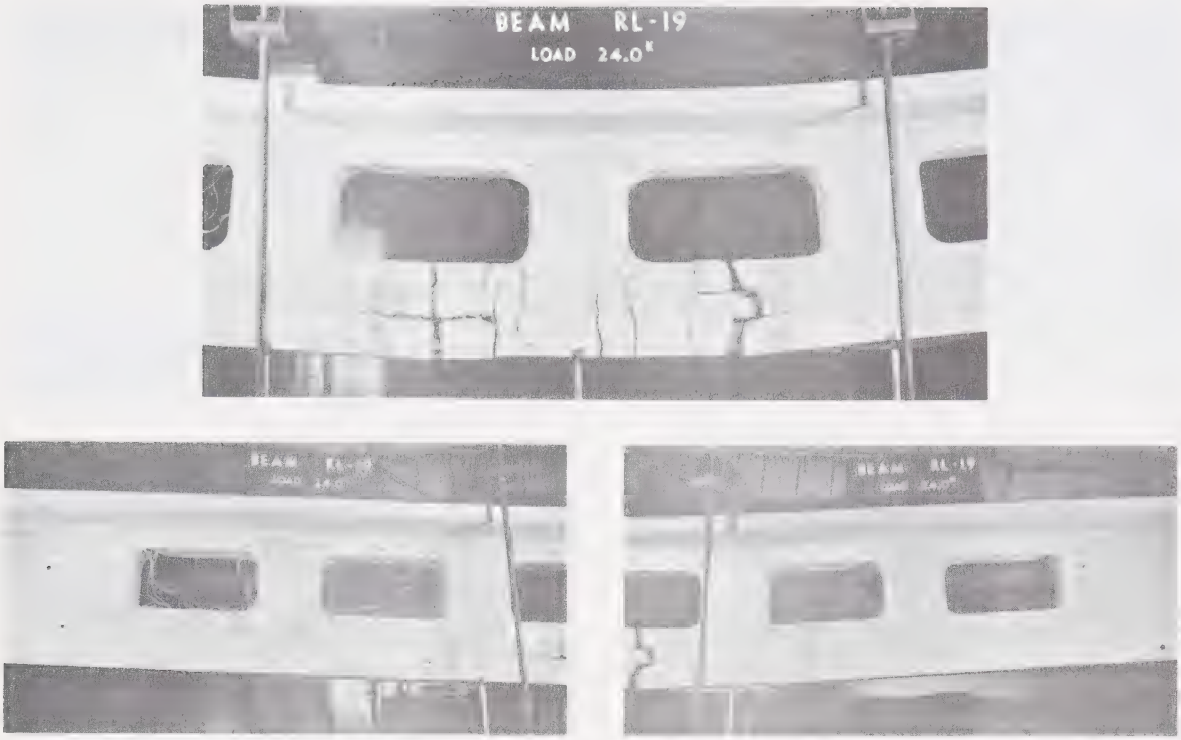


FIGURE 4.127   CRACKING AND FAILURE PATTERN OF BEAM 19-16-6

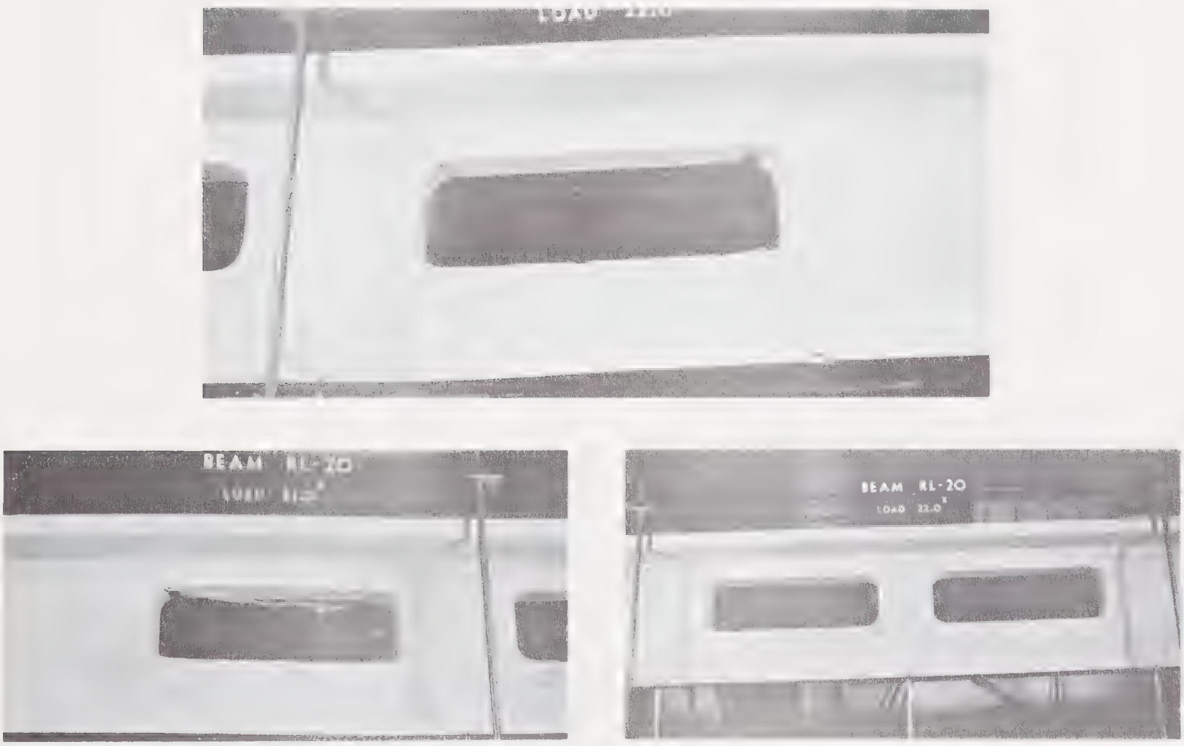


FIGURE 4.128   CRACKING AND FAILURE PATTERN OF BEAM 20-26-5

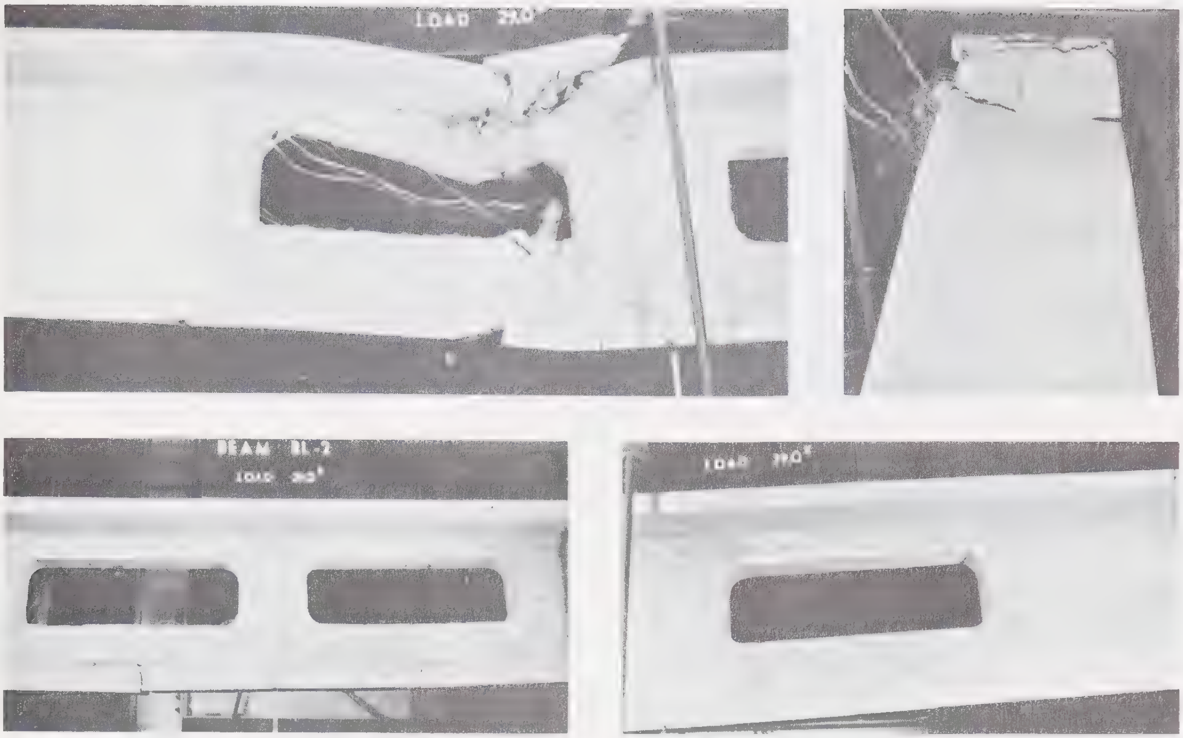


FIGURE 4.129 CRACKING AND FAILURE PATTERN OF BEAM 21-26-5

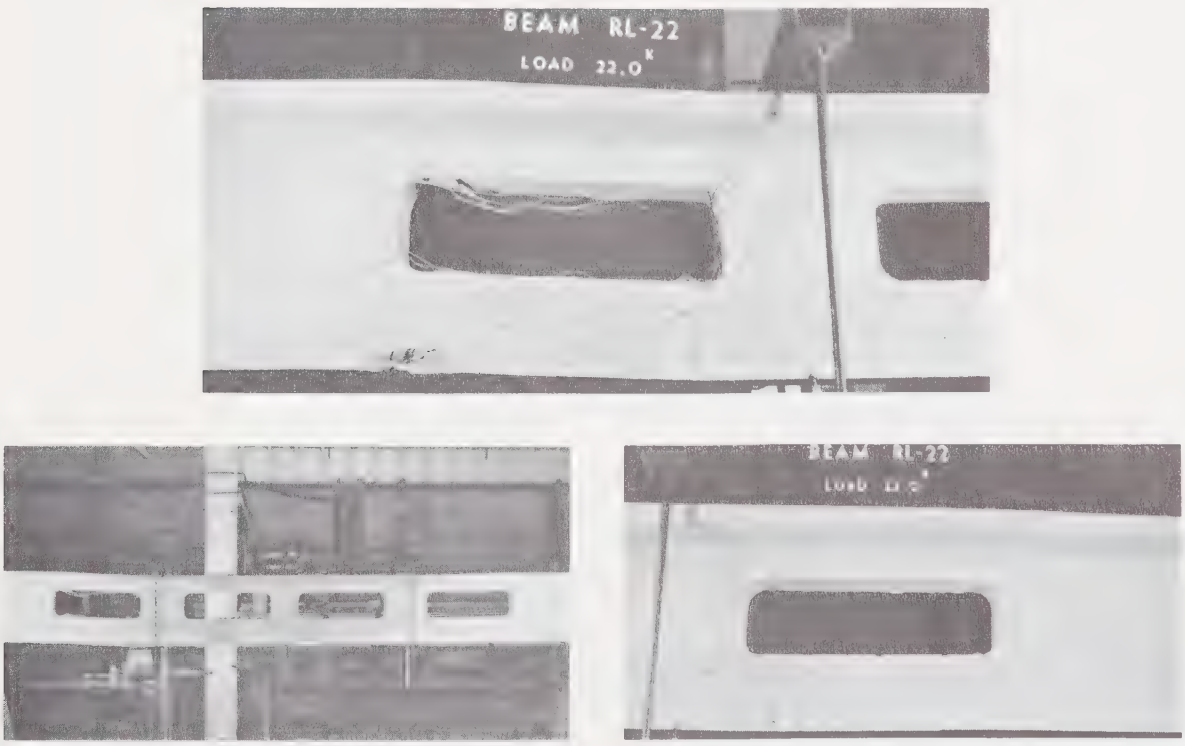


FIGURE 4.130 CRACKING AND FAILURE PATTERN OF BEAM 22-26-5

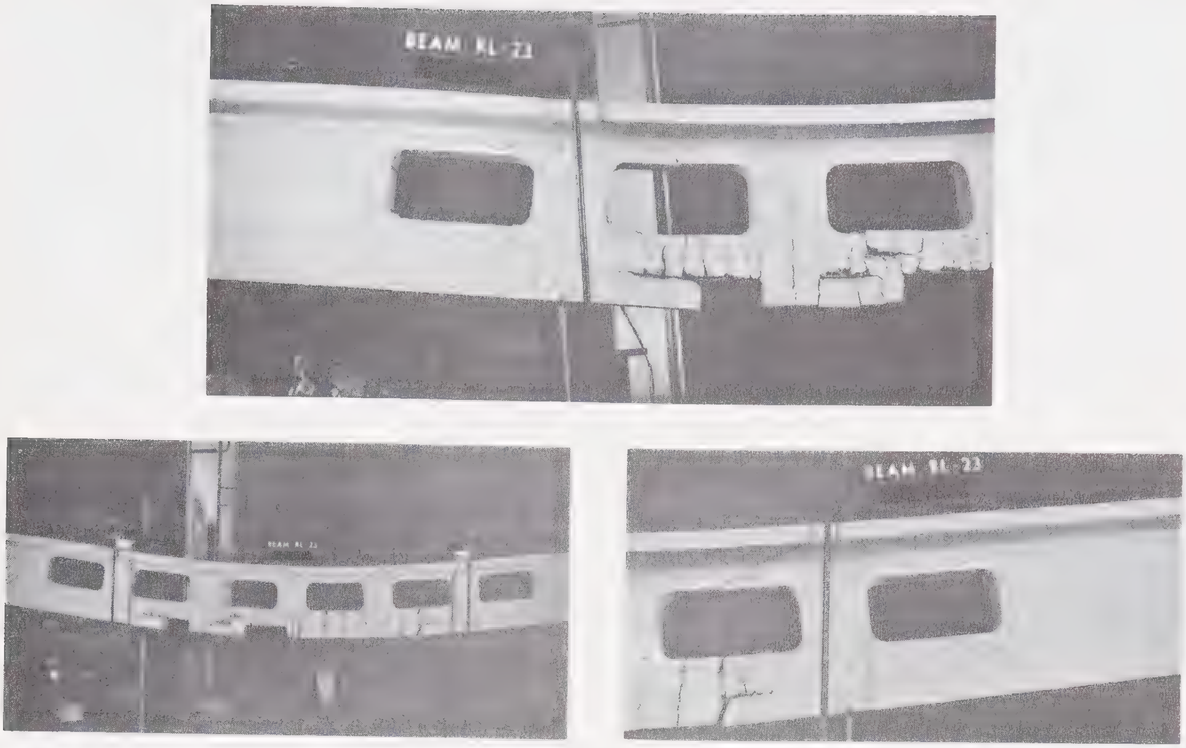


FIGURE 4.131 CRACKING AND FAILURE PATTERN OF BEAM 23-16-4

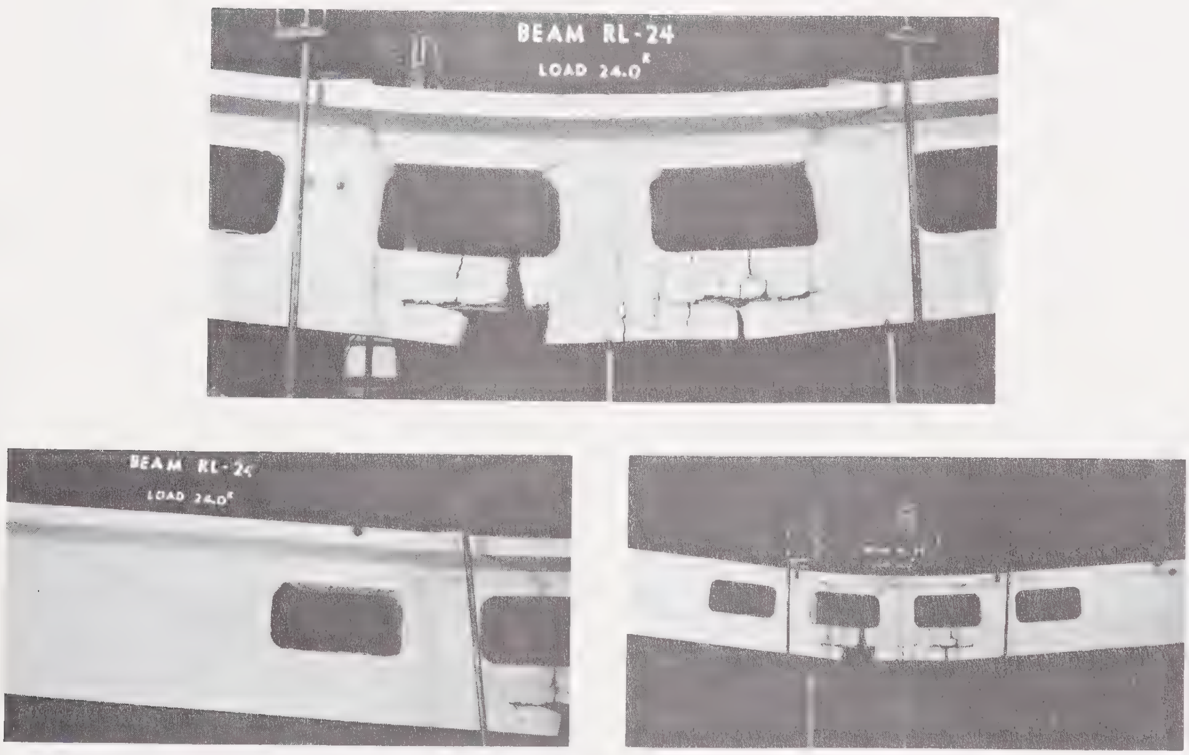


FIGURE 4.132 CRACKING AND FAILURE PATTERN OF BEAM 24-16-6

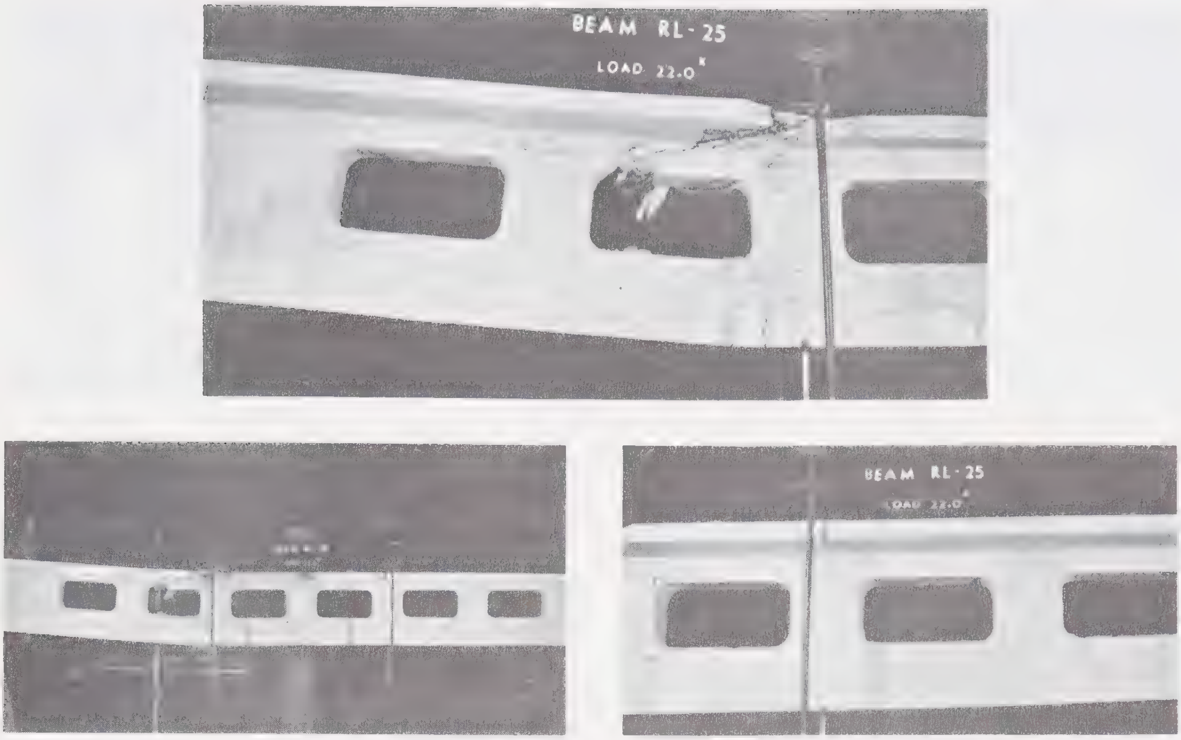


FIGURE 4.133 CRACKING AND FAILURE PATTERN OF BEAM 25-16-6

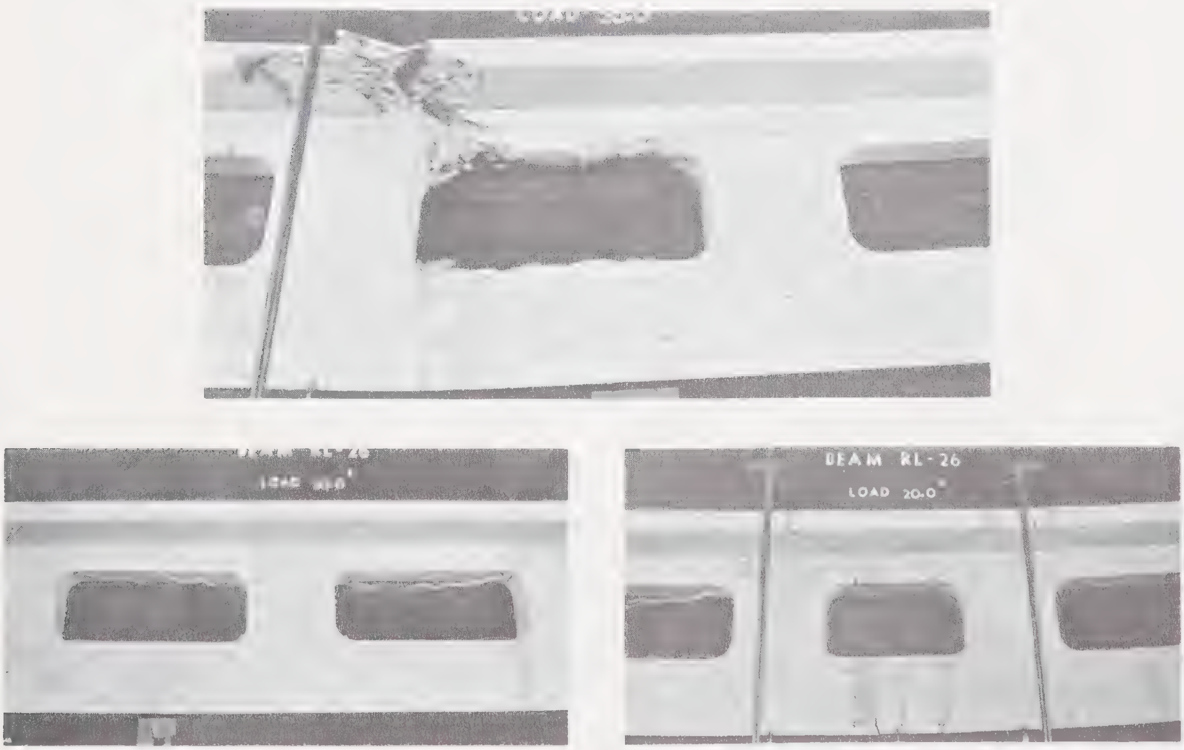


FIGURE 4.134 CRACKING AND FAILURE PATTERN OF BEAM 26-21-7

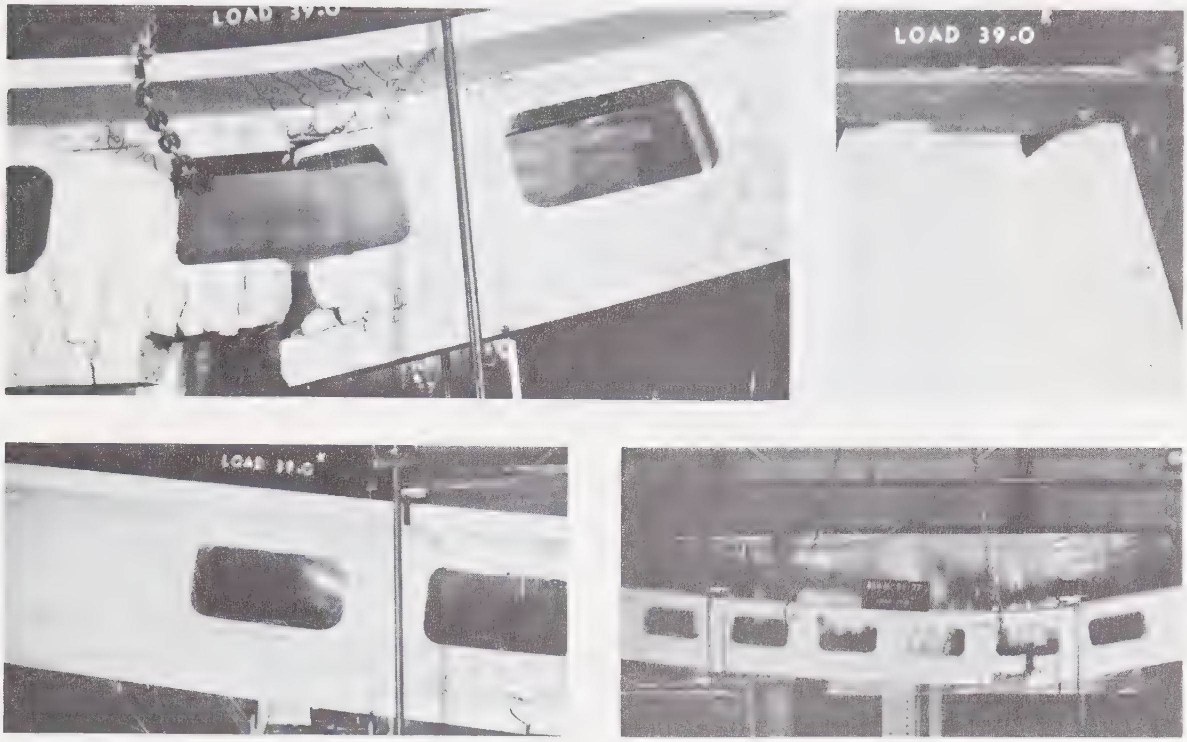


FIGURE 4.135 CRACKING AND FAILURE PATTERN OF BEAM 27-16-4

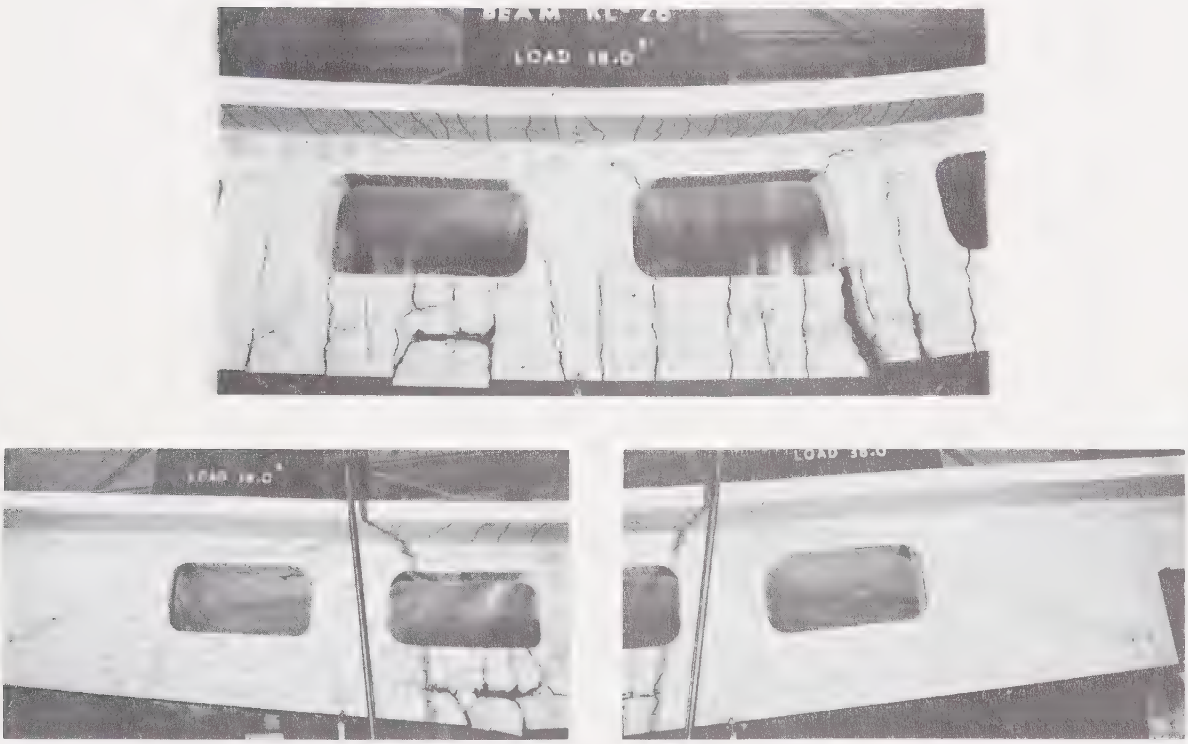


FIGURE 4.136 CRACKING AND FAILURE PATTERN OF BEAM 28-16-4

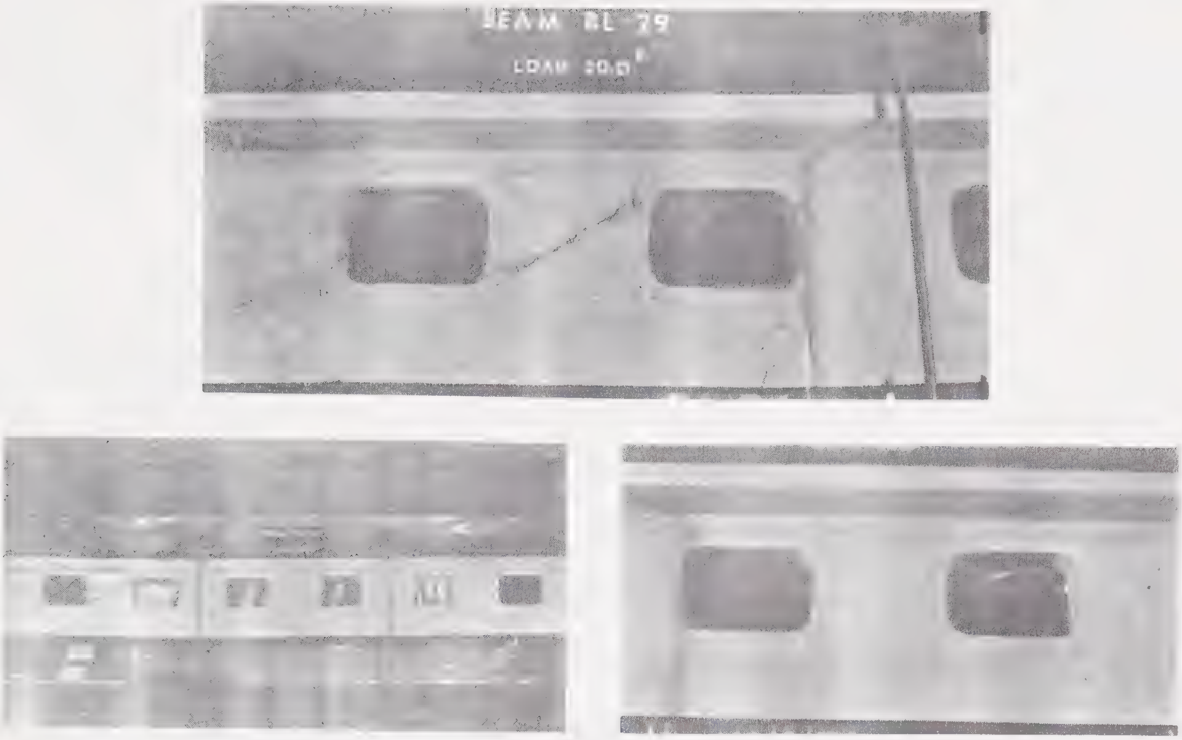


FIGURE 4.137   CRACKING AND FAILURE PATTERN OF BEAM 29-12-6

FIGURE 4.138   CRACKING AND FAILURE PATTERN OF BEAM 30-21-7

CHAPTER 5

DISCUSSION

5.1 Introduction

This chapter presents a discussion of the thirty beams tested in this series and the effects of major variables involved. The major variables were reinforcement in the region of the holes, beam geometry, loading, flexural capacity and material properties. These variables and their effects on beam behaviour are discussed in Section 5.2. The behaviour of individual beams is discussed more completely in Section 5.3. Here the beams are grouped according to failure mode. The discussion in Section 5.3 centers around the behaviour in relation to the results presented in Chapter 4 and the failure mode. Because the cracking behaviour was similar for all the beams of this series a general discussion is presented in Section 5.4.

5.2 Beam Parameters

The major variable of this study was the reinforcement required in the region around a hole. Other parameters such as beam geometry, loading conditions and flexural capacity were varied to place different demands on the reinforcement in the region around a hole. Regarding the geometrical changes, the span length was varied to facilitate casting two beams in the available 40 feet of forms. The material properties varied in a random manner.

The reinforcement in the region around the holes was grouped into four types:

1. Post reinforcement.
2. Solid shear span shear reinforcement.
3. Strut shear reinforcement.
4. Strut flexural reinforcement.

The main post reinforcement consisted of double legged No. 2 or 3 stirrups as shown in Figure 3.3 set vertically or inclined at 45°. Supplementary post reinforcement consisting of horizontal stirrups or inclined bars was used in some cases along with the vertical double legged stirrups. Increasing the number of vertical or inclined double legged stirrups increased the failure load of the posts as was previously noted by LeBlanc (6) and Sauve (11) in their studies. Inclined stirrups gave the posts higher capacity than vertical stirrups and also reinforced the top strut against shear near the load end of the strut. Supplementary post reinforcement increased the post strength. The response of post reinforcement to applied loads was monitored with the use of electrical resistance strain gages. The response of these gages is recorded graphically in Figures 4.29 to 4.38 for the main stirrups and for the supplementary post reinforcement in Figure 4.88.

The stirrups in the solid shear spans had the same shape as the post stirrups and were fabricated from No. 2 or 3 bars. No failures occurred in this region although some of the beams were 20 percent under designed according to the ACI code (1). Several reinforcing arrangements were used. In the first 16 beams cast, No. 3 stirrups were used spaced at 6, 8 or 12 inches and set vertically or inclined

at 45°. For the next 6 beams, three No. 3 stirrups were placed beside hole 1 then spaced at 15 inches, the maximum allowable for the concrete section according to the ACI code (1). The remaining eight beams were cast with 2 or 3 No. 3 stirrups beside hole 1 followed by No. 2 stirrups spaced from 6 to 15 inches according to Section 11.5.2 of the ACI code (1). The lighter shear reinforcement allowed the shear cracks to open wider but as was noted earlier no failures occurred in this region of the beams. The response of the gages at stirrup locations 7 and 8 in this region are plotted in Figures 4.39 to 4.47.

The shear reinforcement in the top and bottom struts of the beams of this series eliminated the most common failure of the test series by LeBlanc (6) and Sauve (11) from all but 5 beams of this series. The strut shear reinforcement used in this series consisted of closed No. 2 stirrups as shown in Figures 3.2 and 3.3 set vertically or inclined at 45°. The vertical stirrups in the top strut were spaced from 1.19 to 4.00 inches and the inclined stirrups were spaced at 3.50 inches. The bottom strut stirrups set vertically had spacings from 1.06 to 4.00 inches and the inclined stirrups were spaced at 3.50 inches. The strut stirrup spacing for each beam is given in Table 3.1. Decreasing the spacing of the strut shear reinforcement increased the shear capacity of the struts. Sloped post stirrups and diagonal supplementary post reinforcement increased the strut shear capacity also. The response of the strut shear reinforcement to the applied loads at gage locations 9 to 24 is plotted in Figures 4.48 to 4.87.

With higher strut shear capacity in the struts, strut flexural

failures became the limiting factor in six beams of this series. The strut flexural reinforcement was placed in four layers over the depth of the section. At the top of the top strut, four No. 2 or 3 bars were used. In the other three levels, i.e., bottom of the top strut, top of the bottom strut and at the bottom of the bottom strut, the strut flexural reinforcement consisted of two No. 2, 3, 4 or 5 bars. Increasing the area of strut flexural reinforcement increased the load at which strut flexural failures occurred for similar loading conditions and hole lengths. Longer holes required more strut flexural reinforcement. Bond appeared to be a problem for the strut flexural reinforcement located at the bottom of the top strut where the stresses varied from yield in tension to yield in compression over the length of the struts and across the posts. Gages were placed on the strut flexural reinforcement of the last 16 beams cast. The response of these gages is recorded graphically in Figures 4.87 to 4.102 for gage locations 25 to 40.

The beam geometry was varied: (i) to place different demands on the reinforcement in the region of the holes and (ii) to facilitate casting. The changes in geometry in the first category are in relation to post and hole length and hole spacing and shape, and the second category is related to the span length. Several of the beams' geometric properties were constant throughout all the tests. The cross sectional area of the solid beam was 114.25 square inches and the cross sectional area of a hole was 32.0 square inches reducing the area 28 percent. The gross moment of inertia of the solid concrete section was 4583 in^4 and through a hole 4369 in^4, a reduction of 4.9 percent.

Increasing the length of the holes placed higher demands on the strut flexural reinforcement and the post reinforcement. Increasing the post length for the same hole spacing increased the load at which the posts cracked and the failure load of the beams failing in post shear.

Altering the shape of the holes from a rectangle to a parallelogram facilitated the placing of sloped stirrups in the posts and increased the beam failure load. Hence such an arrangement was more efficient as pointed out by LeBlanc (6).

The clear span was 20 feet for the first 16 beams and 16 feet for the remaining 14 beams. The shorter span was used to facilitate casting two specimens at once in the available 40 feet of forms. Shortening the span length decreased the length of the pure moment region and thus the deflections for the same shear span length.

The loading was varied to place different demands on the reinforcement in the region around the holes by varying the applied shear and moment along the beam length. Seven point loading was applied to two beams and the remaining 28 beams had symmetrical two point loads with shear spans of 4, 5, 6 or 7 feet.

The seven-point loading was used to simulate a uniformly distributed load without the extra strut moments due to transverse strut loads. One of the beams with this loading arrangement failed in flexure and the other in post shear.

Of the two point loadings the 4 ft shear span resulted in the most severe shear conditions placing high demands not only on the strut shear and flexural reinforcement but also on the shear reinforcement in the solid shear spans. Eight beams were loaded with this load arrangement; four failed in flexure, three in strut shear and one in strut flexure.

The two point loading with 5 ft shear spans produced high demands on the strut flexural reinforcement because it was used on beams with holes 26 inches long. All of the beams with this loading failed in strut flexure.

Two point loading with 6 ft shear spans was used in 15 of the tests. This loading was used to study the behaviour of a post in the shear span and its effect on adjacent holes. This load arrangement produced the maximum horizontal shear on a post because the change in moment between the centerlines of the holes was maximum. Of the 15 beams tested using this load arrangement some beams failed in each of the four failure modes observed in this series; five failed in flexure, seven in post shear, two in strut shear and one in strut flexure.

Two beams were tested with a two point loading and 7 ft shear spans. The main reason for this loading was to study the effect of increasing the hole length and spacing on the struts and posts of the shear spans. One of these beams failed in strut flexure and one in post shear.

The flexural capacity is the last parameter discussed which

was varied to place different demands on the reinforcement in the region of the holes. Increasing the flexural capacity was accomplished by increasing the number of prestressing strands. Seven of the first ten beams were cast with four 3/8 inch diameter seven-wire prestressing strands and all of the other twenty-three beams had five strands. Two of the beams with four strands and eight of the beams with five strands failed in flexure.

Variations in material properties have some effect on the beam capacity but were not considered as major variables. The concrete used had an average compressive strength of 5554 psi with a maximum variation of 14 percent. This change in compressive strength resulted in a calculated change in the flexural capacity of less than 1.0 percent. The average splitting strength was 409 psi and varied from 301 to 548 psi. The modulous of elasticity was calculated for the concrete in the first five beams and found to average 3.26×10^6 psi with $E/\sqrt{f_c'} = 44{,}000$, considerably less than the ACI (1) recommended value of 57,000. The mild steel reinforcement had yield strength of 44.3 or 33.8 ksi for No. 2 bars straight and stirrups, 51.4 ksi for straight No. 3 bars, 55.2 ksi for No. 3 stirrups, 58.2 ksi for No. 4 bars and 54.2 ksi for No. 5 bars. The properties of the prestressing strand were assumed to be constant as only one reel was used. Properties of all materials used are given in Appendix A.

5.3 Beam Behaviour

In this section the behaviour of individual beams, grouped according to failure mode, are compared and discussed. There were four

failure modes observed in this series; (i) flexural, (ii) post shear, (iii) strut shear and, (iv) strut flexure. These failure modes are defined in relation to this series at the beginning of the discussion of each group of beams exhibiting that failure mode. The beams in each group are sub-grouped according to the applied loading and discussed in relation to the test results presented in Chapter 4 and the failure mode.

The beams failing in flexure are treated as control beams and the behaviour of other beams are compared to them.

5.3.1 Beams Failing in Flexure

Flexural failure is a failure due to applied moment alone, resulting in the failure of the tension reinforcement or crushing of the concrete. In this series all flexural failures were due to rupture of one or more of the prestressing strands. This type of failure is desirable in that it is extremely ductile. The maximum deflections varied from 5.27 to 10.28 in. giving adequate warning of the pending failure. The flexural capacity of the beams was accurately calculated using Section 18-7 of the ACI Code (1) with $\phi = 1$. The actual flexural capacity ranged from 1.03 to 1.16 of the calculated capacity. In this series 10 of the beams failed in flexure under three of the five different loadings. One beam (8-16-7L)* with a seven-point load failed in flexure. Nine beams with two-point loads also failed in flexure, four with 4 ft. shear spans (18-16-4, 23-16-4, 27-16-4, and 28-16-4) and five with 6 ft. shear spans (10-16-6, 12-16-6-P, 14-12-6, 19-16-6, and 24-16-6). The two loadings without flexural failures were the two-point loadings with 5 and 7 ft. shear spans which were used in conjunction with larger holes

* For beam number code, refer to Table 3.1, page 19.

26 and 21 inches long respectively.

Beam 8-16-7L, the only beam with a seven-point loading to fail in flexure, had 8 by 16 inch holes spaced at 2 feet and 4 3/8 inch 7-wire prestressing strands as the primary flexural reinforcement. The behaviour of this beam was that of a typical under-reinforced prestressed concrete T-beam as evidenced by the moment-deflection curve, cracking patterns, centerline concrete strain distribution and the moment-strain relationships for the prestressing strands.

Failure occurred by rupture of all four prestressing strands at the beam centerline at a load of 5.75 k per jack. The maximum applied moment was 1578 in-k or 1.16 of the theoretical ultimate moment high for three reasons: (1) theoretical calculations assume concrete failure, (2) the concrete in the high moment region is confined by the loading apparatus, (3) the short length of the high moment region.

The moment deflection diagram indicates that beam 8-16-7L is less stiff than beam EL-1 (Ref. 6, LeBlanc's control beam without holes) above a moment of 1000 in-k. This is because though beam EL-1 had the same loading, span length, and primary flexural reinforcement, it had two No. 3 supplemental longitudinal reinforcing bars running the full length of the beam in the bottom of the tee. This extra flexural reinforcement increased the theoretical flexural capacity approximately 224 in-k.

The photographs of the cracking patterns show the flexure cracks extending 0.75 inches into the top flange at the ultimate load. The centerline concrete strain distribution at the same load indicated

the neutral axis was 1.25 inches from the extreme compression fiber. The demec gages on the top surface of the flange were shifted 8 inches from the beam centerline to accommodate the load at the beam centerline but they all recorded maximum compressive strains greater than 0.003 in/in.

The measured strain in the prestressing strand at the beam centerline indicated the stress in the strands was within 5.0 percent of the manufacturer's tested ultimate tensile stress of 275.6 ksi. The strain in the prestressing strand decreased at gage locations further from the beam centerline because of the reduced bending moment and the supplemented longitudinal reinforcement provided. At gage location 4, 7 feet from the centerline, the total force in the prestressing strand was 61 percent of the ultimate tensile strength just slightly above the initial prestressing force. The maximum moment at this section was 72 percent of the moment at the beam centerline.

The eleven gages on the shear reinforcement placed around holes 1 and 2 showed generally the shear reinforcement contributed little to the ultimate strength of the beam. Only one gage, gage 6 in post 1, indicated strains above the yield strain. The horizontal shear on post 1 under a seven-point loading is not as severe as the post load on post 1 under a two-point load with a 6 feet shear span. The seven-point load does, however, produce the most severe load on post 2. The strain in this post remained below the yield strain and the crack through the post remained small. At gage 7 on the long stirrups near hole 1 in the solid shear span the strain reached 57 percent of the yield strain. The gages on the stirrups in the top and bottom struts all showed strains less than

26 percent of the yield strain and the shear cracks in the strut remained small. This is because of the high moments and low shears associated with the seven-point loading.

The four beams with 4 feet shear spans that failed in flexure all had the same geometry and very similar reinforcement. The holes were 8 by 16 inches located at 2 feet on the center and the clear span in all cases was 16 feet. The primary flexural reinforcement was 5, 3/8 inch 7-wire prestressing strands running the full length of the beam. The strut flexural reinforcement in the shear spans consisted of four No. 3 bars and six No. 5 bars. The No. 5 bars were anchored in the solid shear span beyond the support for beams 18-16-4 and 16 inches beyond hole 1 in the other three beams. In the pure moment region, the bars were cut off at the centerline of hole 2. The solid shear spans were reinforced with three No. 3 stirrups beside hole 1 and then No. 3 stirrups were spaced at 15 inches for beam 18-16-4 and No. 2 stirrups at 6 inches for beams 23-16-4, 27-16-4 and 28-16-4. The spacings for the stirrups in the top struts varied from 1.25 to 2.50 inches and in the bottom strut from 1.06 to 2.13 inches.

The general behaviour of each of these beams was that of a typical under-reinforced prestressed concrete T-beam; all failed by rupture of the prestressing strands. Three of the beams failed at a load of 38 kips per jack with an ultimate moment of 1824 in-k or 1.09 times the theoretical ultimate capacity. Beam 27-16-4 failed at 39 k per jack or 1.11 times the theoretical ultimate load. The higher ultimate capacity of this beam is the result of the north shear span being less than

4 feet because a nut, used in leveling the bearing plates that rest on the knife edge of the roller support, was inadvertently left in place. The maximum possible reduction in the shear span length was 3 inches. Using this asymetrical loading, the maximum applied moment was 98 percent of the moment corresponding to the symmetrical loading and the shear on the north shear span was increased by 2 percent. The cracking pattern and deflections of the beam at the load point also indicated the loading was not symmetrical. However, as the difference was small, all the results are based on the symmetrical loading.

The moment-deflection diagrams for the beams with 4 foot shear spans failing in flexure were all similar. Up to a moment of 720 in-kips they are linear, for higher moments the slope decreases to almost zero near the ultimate moment. The maximum deflection for these beams was between 7 and 9 inches.

The concrete strains at the beam centerline indicated the neutral axis was approximately 2 inches below the extreme compression fiber at the ultimate load at which time the flexural cracks were within 1.0 inches of the top of the beam. The maximum recorded strain in the concrete at the beam centerline was over 0.003 in per in for each of the beams. The distribution of the strain over the depth of the section was reasonably linear with a few irregularities at an applied moment of 1000 in-k.

The moment-strain curves for the prestressing strand at the beam centerline (gage 1) had the same shape as the moment-deflection curves, being linear up to a moment of 720 inches and the slope at higher

loads decreasing to almost zero at the ultimate load. At the centerline of hole 1, 5 feet from the beam centerline, the strain indicated by gage 4 was less than 50 percent of the strain in the strand at the beam centerline for the same applied moment.

The 4 foot shear span produced the greatest demands on the shear reinforcement except for the post reinforcement in post 1. The reinforcement in post 1 for these four beams consisted of four double-legged No. 3 stirrups. The theoretical horizontal shear developed in the post was 76 percent of the horizontal shear in the same post for beams with 6 foot shear spans. This coupled with the fact that the axial load was high in post, due to the applied load, made post 1 less critical under this loading. The gages in post 1 at gage location 6 all exhibited the same behaviour to the post cracking load, that is, very small compression. The load at which the post cracks formed in post 1 varied from 16 to 20 k. As the post cracked, the strains changed from a small compression to tension over 0.0005 in/in. In beams 18-16-4 and 28-16-4 the strain continued to increase slowly to a load of 30 k; for higher loads the strain decreased slightly to failure. The strain in the post stirrups in beam 23-16-4 increased more rapidly than in the others but the gage became inoperative at a load of 26 kips at a strain level below yield. The stirrup of beam 27-16-4 was the only one to reach the yield strain and did so at a load of 32 kips. The post reinforcement was adequate since the post cracks remained small.

The reinforcement in the solid shear span also proved adequate and the cracks, though larger than in the posts, remained small. The

three stirrups beside hole 1 in all the beams with 4 ft shear spans behaved in a similar manner having small tensile strains up to about 15 kips then increased more quickly and approached a reading near the yield strain at the ultimate load. The stirrups 0.625 in. from the hole showed just slightly less strain than those 3.50 in. from the hole for the same load. The slope of the load strain diagrams for these stirrups generally increased above a load of 30 kips indicating yielding at other locations along the stirrups. Beams 23-16-4, 27-16-4 and 28-16-4 also had gages placed on stirrups in the solid shear span 15 inches from the edge of hole 1 near the cutoff of the strut flexural reinforcement. Only in beam 28-16-4 did this gage (gage 8A) indicate a significant strain. In this beam the strain was small to a load of 30 kips after which the strain increased rapidly to the yield strain at a load of 36 kips.

The strains measured in the stirrups of the top struts indicated the stirrups were adequate for all spacings. The strains were highest in the first stirrups near the load at gage 13. For this gage in beam 28-16-4 the strain reached 97 percent of the yield strain. The load strain diagrams for this gage in beams 23-16-4 and 27-16-4 were similar to that of 28-16-4 but the strains were slightly smaller due to the closer stirrup spacing. The general shape of the load strain curve for gage 13 in these three beams was an "S" curve having small strains to a load of 12 kips then increasing at an increasing ratio to a load between 19 and 21 kips above which the strain increased slowly to the ultimate load. In beam 18-16-4 the behaviour of gage 13 was quite different and indicated the yield strain was reached at a load of 18 kips. This was most likely due to a malfunction of the gage as the plotted

curve was much different than for the other beams while the cracking patterns were similar for all four beams of the group. Beam 18-16-4 was the only beam in this sub-group with a gage at the center of the top strut (gage 19) and the load strain curve for this gage correlated well with the cracking pattern. The recorded strain was small to a load of 20 kips when a web shear crack appeared; the strain at higher loads increased in a regular manner to yield at a load of 32 kips.

In the stirrups of the bottom strut the strains were highest for gage 23 at the centerline of the hole. Although three of the four gages at this location indicated yielding before the ultimate load was reached, the rate of strain was relatively constant up to the yield strain and beyond and the shear cracks remained small. The load strain plots for this gage indicated a small compressive strain up to a load of 12 to 15 kips when the strain jumped to tension and increased slowly to the ultimate load. The stirrup spacing had the expected result on the recorded strain, that is, the largest spacings led to the largest strains. The difference was, however, less than the change in spacing. In beam 27-16-4 with the smallest spacing, 1.06 inches, the strain reached 97 percent of the yield strain at a load of 38 kips while beams 18-16-4 and 23-16-4 with 1.50 inch spacing reached yield at 32 and 34 kips, respectively. Beam 28-16-4 with 2.50 inch spacing, double that of beam 27-16-4, reached the yield strain at a load of 26 kips.

The top strut flexural reinforcement consisted of four No. 3 bars in the top and two No. 5 bars in the bottom. The strain measurement taken clearly shows Vierendeel type of strut behaviour with the

strain in the horizontal bars changing from yield in tension to yield in compression along the length of the struts. This change in stress along the length of the struts produced large bond stresses and in three of this group there appears to have been some slip.

In beam 18-16-4 gages were placed on both the No. 3 and No. 5 bars at both ends of both struts. Gage 29 on a No. 3 bar in the top of the strut near the load indicated the compressive strain increased at a fairly uniform rate to 95 percent of the yield strain at the ultimate load. Directly below gage 29, gage 30 on one of the No. 5 bars, the strain increased in tension slowly to 54 percent of the yield strain at the ultimate load. At the reaction end of the top strut gage 31 on a No. 3 bar in the top of the strut indicated that the tensile strain increased slowly to 64 percent of the yield strain at failure. At gage 32, directly below gage 31, the strain increased in compression to a load of 22 kips; for all higher loads the compression decreased and at failure the strain was 50 percent of the yield strain in tension indicating that bond failure had occurred. In the other three beams of this group, only two gages were mounted on the strut flexural reinforcement of the top struts, gages 30 and 32 on the No. 5 bars. The load strain curve for gage 30 in beam 28-16-4 was similar to that of beam 18-16-4. In beams 23-16-4 and 27-16-4, the strains were larger and reached the yield strain at loads of 34 and 36 kips, respectively. Gage 32 for beam 27-16-4 and 28-16-4 indicated the strains were similar to those for beam 18-16-4 but the decrease in compressive strain started at a load of 26 kips. In beam 23-16-4 gage 32 showed a uniform increase in compression to 95 percent of the yield strain at the ultimate load.

The measured strains in the bottom strut also indicated Vierendeel truss behaviour but not as strongly as in the top strut because the compressive strains of the strut flexure are masked by the tension due to the beam moment. Beam 18-16-4 had gages at locations 37, 38, 39 and 40 while the other three beams had gages at locations 38 and 39 only. Gages 37 and 40 were in the compression zone for the strut flexural moment, that is, at the top near the load and at the bottom near the reaction. However, these gages registered tensile strains indicating the compressive force was limited to a small area of concrete beyond the reinforcement. This is substantiated by the strut cracking pattern. The load strain curves for gages 38 and 39 were similar for each of the four beams in spite of the different behaviours observed in the top struts. The strain in these gages increased in tension to the failure load with the yield strain being reached at a load between 28 and 30 kips for gage 38 and between 30 and 32 kips for gage 39.

There were five beams that failed in flexure with 6 foot shear spans; all failed by rupture of the strands at loads between 103 and 108 percent of their theoretical flexural capacities. Although these five beams all failed in a similar manner, there were major differences in the reinforcement around the holes, in the primary flexural reinforcement, in the span length, and in the size and shape of the holes.

The behaviour of each of these beams was that of a typical under-reinforced prestressed concrete T-beam, with large deflections and high strains, in the concrete in compression, and in the prestressing strands in tension.

The moment deflection curves for these beams fell into three groups depending on flexural reinforcement and span length. The first group was made up of beams with four prestressing strands as primary tension reinforcement and 20 foot clear span. Beam 10-16-6 was the only beam of this series in this group. J. Sauve's (11) control beam (JS-1) also had the same span and tension reinforcement but had no holes. The deflection of beam JS-1 was slightly less than for beam 10-16-6 throughout all stages of loading with the largest differences occurring between moments of 720 and 1200 in-kips when the cracks in beam 10-16-6 indicated the neutral axis was between the top of the bottom strut and the bottom of the top strut. Beams 12-16-6-P and 14-12-6 with 20 foot clear spans and 5 prestressing strands made up the second group. The moment deflection curves for these beams were similar being linear up to a moment of 720 in-kips with the slope at higher loads decreasing to the ultimate load with the maximum deflection over 8.0 inches. Beams 19-16-6 and 24-16-6 with 16 foot clear spans and 5 prestressing strands form the third group. The load strain curves for these two beams were almost identical and had the same shape as those of the second group (12-16-6-P and 14-12-6) but the strain was slightly smaller due to the shorter span length.

For the five beams failing in flexure, the measured strain at the beams' centerline at the penultimate load indicated the maximum compressive strains in the concrete were between 0.0027 and 0.0034. From the strain distribution at the same load, the neutral axis was located between 2 and 2.5 inches below the extreme compression fiber. The cracking patterns at the ultimate load indicated the neutral axis

was between 1.0 and 2.5 inches below the top of the beam.

The moment-strain curves for the prestressing strands in the pure moment region for gages 1 and 2 had the same shape as the load deflection curves. Increasing the number of prestressing strands decreased the strain but the changes in geometry did not. In the shear spans gages 3 and 4, on the prestressing strands, indicated the strain decreased towards the support and that the strain was less than at gages 1 and 2 for the same applied moment.

Gage 5 on the post reinforcement in post 2, below the load, indicated the post reinforcement was more than adequate. Beam 10-16-6 had 3 vertical double legged No. 3 stirrups while beam 19-16-6 and 24-16-6 had 4 vertical stirrups and beams 12-16-6-P and 14-12-6 had 3 inclined stirrups in post 2. The measured strains were between 200 and 650 micro in per in and did not appear to be affected by the different geometries on the reinforcing arrangements.

For beams having 6 ft shear spans and one post in the shear span, the horizontal shear on post 1 was severe. This load and beam arrangement was used in seven of the nine beams that failed in post shear. Several different concrete layouts and reinforcing arrangements were used for the posts in the shear spans of the five beams that failed in flexure with 6 ft shear spans. For this reason, the posts and their reinforcement will be discussed separately for each of the five beams of this group.

Beam 10-16-6 had vertical posts 8 inches long reinforced with

three vertical double legged No. 3 stirrups and supplementary reinforcement consisting of two No. 5 bars at 45°. Gage 6 on the vertical stirrups was inoperative. Gage 41 on the supplementary post reinforcement had an irregular load strain plot that indicated the axial load in the No. 5 bars, at the load where the bars reached the yield strain, was higher than in other beams with only sloped stirrups indicating a possible malfunction. Yielding occurred at a load of 16.5 kips but the cracks through the post remained small to failure. Beam 12-16-6-P and beam 14-12-6 both had three inclined double legged No. 3 stirrups in post 1 but 12-16-6-P had parallelogram shaped openings 16 inches long and inclined post while 14-12-6 had rectangular openings 12 inches long and vertical posts 12 inches long. Gage 6 in beam 12-16-6-P indicated the stirrups reached the yield strain at a load of 22 kips while in beam 14-12-6 the maximum recorded strain was 85% of the yield strain at the ultimate load. Beam 19-16-6 had vertical posts 8 inches long with four vertical double legged No. 3 stirrups as the main reinforcement in post 1 and seven closed No. 2 stirrups set horizontally along the height of the posts as supplementary reinforcement. Gage 6 on the vertical stirrups indicated little strain up to a load of 4 kips; at higher loads the load strain relationship was linear to the penultimate load when the strain reached 98 percent of the yield strain. Gage 41 on the horizontal stirrups had a similar load-strain curve which was small to a load 4 kips and increased in a linear manner for higher loads to the yield strain at the penultimate load. Whereas in all the other beams with 6 ft shear spans the centerline of the first hole was 3 feet from the support, in beam 24-16-6 the centerline of the first hole was

5 feet from the support, therefore, there was no post in the shear spans of this beam.

The plotted load-strain curves for gage 7, on the first stirrup in the solid shear span, were similar for beams 10-16-6, 12-16-6-P, 14-12-6 and 19-16-6 in spite of the different reinforcement arrangements. Generally, the strain was small up to a load at 7 kips and the strain at higher loads increased in a linear manner with yielding occurring at loads between 18 and 25 kips. The strain for gage 7 in beam 24-16-6 was much less than in other beams with 6 foot shear spans but similar to the strains for gage 7 in beams with 4 foot shear spans. Closer to the supports at gage locations 8 and 8A, the maximum recorded strains were less than 650 micro in per in.

A variety of shear reinforcing arrangements were used in the top and bottom struts of the beams with 6 foot shear spans that failed in flexure. Beam 10-16-6 had closed vertical stirrups spaced at 3 1/2 inches in both top and bottom struts. The strut shear reinforcement in beam 12-16-6-P consisted of closed No. 2 stirrups set at 45° and spaced at 3 1/2 inches in both top and bottom struts. No strut shear reinforcement was provided in beam 14-12-6. In beams 19-16-6 and 24-16-6, the strut shear reinforcement consisted of closed vertical No. 2 stirrups in the top strut spaced at 1 7/8 and 3 inches, respectively, and in the bottom strut spaced at 1 1/2 and 1 5/8 inches, respectively.

For beam 10-16-6, the strain in the top struts were larger above hole 1 than hole 2 and reached a maximum of 98 percent of the yield strain for gage 14, on the second stirrup from the load end of

the top strut, at a load of 17 kips. The maximum recorded strain above hole 2 was 56 percent of the yield strain. The strains in the bottom strut stirrups were all below 37 percent of the yield strain with the strain slightly larger below hole 2. The anchorage of the supplementary post reinforcement in the top strut above hole 1 and in the bottom strut below hole 2 may have had some effect on the measured strains.

The stirrup strains in the top struts of beam 12-16-6-P were largest above hole 2 with gage 10 on the second stirrup from the load end of the strut indicating the yield strain at the penultimate load. Above hole 1 the maximum recorded strain was 40% of the yield strain. In the bottom struts the strains were also larger below hole 2 with gage 19 at the strut centerline reaching the yield strain at a load of 18 kips while below hole 1 gage 23 reached 95 percent of the yield strain at the ultimate load.

Though there were no strut shear stirrups placed in the struts of beam 14-12-6, the sloped post reinforcement checked the shear cracks that formed near the load ends of the top struts. This beam also demonstrates the high shear capacity of the concrete section under this loading arrangement. The average shear stress on the concrete at the centerline of the hole is 750 psi based on the minimum web width and the beam depth less the height of the hole.

For beam 19-16-6, the strains measured in the top strut stirrups were all below 50 percent of the yield strain and similar above both holes. In the bottom strut the strains were larger below hole 1 reaching

68 percent of the yield strain for gage 23 at the strut centerline, while below hole 2 gage 19 indicated the strain was below 33 percent of the yield strain.

There was no hole 1 in beam 24-16-6 and the strain in the strut shear reinforcement above and below hole 2 reached maximum strains of 58 and 80 percent of the yield strain.

The various strut shear reinforcing arrangements were all adequate even for beam 14-12-6 which had no strut stirrups. The various spacings of the vertical stirrups had little influence on the recorded strains, however, the inclined stirrups of beam 12-16-6-P generally carried more load than the vertical stirrups.

The top strut flexural reinforcement, in all five of the beams with 6 ft shear spans that failed in flexure, consisted of six No. 3 bars. In the bottom struts the supplemental longitudinal reinforcement consisted of four No. 3 bars; two at the top and two at the bottom of beams 19-16-6 and 24-16-6 while beams 10-16-6, 12-16-6-P and 14-12-6 had only two No. 3 bars in the bottom of the struts. Strain gages were mounted only on the strut flexural reinforcement of beams 19-16-6 and 24-16-6.

The gages in the top struts of beam 19-16-6 indicated the formation of a mechanism over both openings with the yield strain being reached; in compression at gage 32, on the bottom of the top strut of hole 1 near the reaction, and in tension at gage 26 on the bottom of the top strut of hole 2 near the load. The other gages in the top strut

indicated strains less than 40 percent of the yield strain. The gages in the bottom struts indicated the mechanism was forming independently below both openings. At the bottom of the bottom strut where both the beam moment and the strut moments produced tension, gage 34 indicated the reinforcement reached the yield strain at a load of 13 kips. The other gages in the tension zone of both the beam moment and strut moment by gages 38 and 39 indicated yielding at a load of 20 kips.

The gages on the strut flexural reinforcement in beam 24-16-6 indicated slightly less strain than for 19-16-6 for the same load. This suggests that strains above hole 2 of beam 19-16-6 were slightly affected by the adjacent post 1 and hole 1. Gage 26 in the top strut of beam 24-16-6 indicated yielding in tension at a load of 22 kips while gage 28 indicated yielding in compression at a load of 20 kips. Gages 34 and 35 in the bottom strut were both in the tension zone of the strut moments and indicated yielding at 15 and 16 kips respectively.

Strut flexure did not appear to have serious detrimental effects on the behaviour of any of the beams with 6 ft shear spans that failed in flexure, in spite of the fact that some yielding in the strut flexural reinforcement did occur.

The holes in the beams which failed in flexure were adequately reinforced so the full flexural capacity of the beams was reached therefore the beams failing in other modes will be compared to them.

5.3.2 Beams Failing in Post Shear

Post shear failure is a failure initiated by the failure of a

post in the shear span which results in the lateral displacement of the top and bottom portions of the post. This increases the effective hole length placing extra demands on the reinforcement in the region around the holes to either side of the post. The strut flexural reinforcement failed in most of the beams of this group and the shape of the beam after failure was the result of a strut flexural mechanism over two openings.

Post shear failure is undesirable because it does not allow the beam to reach its flexural capacity and occurs with little warning. However, this was the most common failure mode observed in the shear spans for all of the beams tested. Below, the forces and the parameters that affect post shear failure are discussed along with the behaviour of the nine beams failing in post shear, namely, beams 1-16-6, 3-16-6, 4-16-6, 6-16-6, 7-16-6, 9-16-6, 13-16-6-P, 29-12-6 and 30-21-7.

Where shear is present, the major forces acting on the posts are (1) horizontal shear due to the change in beam moments between the centerlines of the openings, (2) moments due to the horizontal shear and the strut moments, (3) axial load due to the applied load and the change in proportion to the shear carried by the top and bottom struts from one hole to the next. The change in moments between the centerline of holes 1 and 2, spaced at 2 feet on center in a beam with 6 foot shear spans, produced a horizontal shear equal to 1.66 times the applied shear force. At the ultimate flexural capacity, the shear stress in a post with a 4 by 8 inch cross-section is 1245 ksi or 16.7 times the square root of the average concrete strength for this series. Assuming an

inflection point at the mid-height of the posts, the maximum bending moment produced by this horizontal shear was 159 in-kips.

The prime variable of this series was the reinforcement in the region around the holes. For the beams failing in post shear, the reinforcement in post 1 had the greatest influence on the post's ability to resist the applied forces and the beam capacity. Increasing the total area of vertical stirrups in post 1 for beams with 8 by 16 inch holes failing in post shear increased the beams' load carrying capacity. Inclining the post reinforcement also increased the capacity of the beams. The addition of horizontal stirrups in the posts increased the shear capacity of the post and reduced the strain in the vertical reinforcement. The strains measured in the post reinforcement at gage 6, for the beams failing in post shear, was greater than in beams with the same geometry, post reinforcement and loading that did not fail in post shear. The post reinforcement in all the beams of this series appeared to be adequate to resist the bending moments due to the horizontal shear. The other reinforcement in the region around the holes was adequate in all the beams failing in post shear and in none of these beams was the beam capacity reduced by failure of any of the reinforcement except the post reinforcement. In fact, the strut flexural reinforcement of beam 4-16-6 was more than adequate. A reduced shear capacity in the bottom strut of this beam resulted in a reduced axial compression on post 1 which caused the post to fail at a relatively low load. At this lower load, the top strut flexural reinforcement was sufficient to carry the strut moments of a strut spanning two openings.

and the ultimate capacity of the beam was 1.5 kips higher than the post failure load of 11 kips per jack.

The size and shape of the posts also affected the post behaviour. Larger posts reduced the shear stresses produced by the horizontal shear and increased the post flexural capacity. For beams with one post completely within the shear span, the post cracking load for this post was highest when the length of the post and the post crack were longest. For beams with vertical 8 by 12 inch posts, this crack was 14.42 inches long and resulted in the highest post cracking loads. The crack length of inclined 8 by 8 inch posts was 5.66 inches and resulted in the lowest post cracking loads. The crack length and the post cracking loads of 8 by 8 inch vertical posts fell between these two extremes. The ultimate capacity and the strain in the post reinforcement were also affected by the post geometry. Although comparison is limited, it appears that increasing the horizontal dimension of vertical posts increases the post's ability to resist horizontal shear and reduces the strain in the post reinforcement. Inclined posts had extra moments, due to the post inclination and the axial load, but because of the inclination of the reinforcement at right angle to the post cracks the ultimate capacity of the posts were increased.

The hole spacing was the other geometric parameter affecting the behaviour of the posts. Increasing the hole spacing increased the change in moment between hole centerline and thus the horizontal shear on the posts decreasing the post cracking load and increasing the strain in the post reinforcement. The comparison of the post

cracking loads for different hole spacings was as expected, less for the larger hole spacings both for beams that failed in post shear and those that did not. The comparison of the strain in the post reinforcement at gage 6, though limited because of differences in post reinforcement, indicated the strain increased with hole spacing but only at higher loads. Because of differences in post reinforcement used in the beams failing in post shear, beams failing in other modes are used for the comparison below. Beams 19-16-6, 25-16-6 and 26-21-7 had similar post reinforcement but hole spacings of 24.0, 24.0 and 30.3 inches respectively. The load strain plots for gage 6 in these three beams were almost identical up to a load of 17 kips after which the strain in 26-21-7, with the longer hole spacing, increased more rapidly than in the other two beams to the ultimate load.

The various loading arrangements used on the beams failing in post shear allowed variation in the geometrical properties of the posts and holes.

Increasing the flexural capacity increased the failure load in two cases and decreased it in one case for otherwise similar beams. Beams 1-16-6 and 6-16-6 with 4 3/8 inch 7-wire prestressing strands failed at loads that were 91 and 92 percent of the failure loads for beams 3-16-6 and 7-16-6 which had 5 strands. However, beam 8-16-7L which had 4 prestressing strands failed at 104 percent of the failure load of 9-16-7L which had 5 prestressing strands.

The general behaviour of all the beams which failed in post

shear are similar. Post 1 cracked at a relatively small load (below 37 percent of the theoretical flexural capacity) after which the post reinforcement picked up load and, in most cases, yielded before failure of the post. The post failure drastically changed the beam geometry by more than doubling the effective hole length which led to the formation of a mechanism and failure of the beam. The load at failure was between 66 and 103 percent of the theoretical flexural capacities of the beams.

In beams 1-16-6, 3-16-6, 4-16-6 and 29-12-6, after post 1 cracked the strain in the post reinforcement continued to increase to the yield strain and beyond in some cases before the post failed which initiated the formation of a strut flexure mechanism over the two openings of the shear span. For beams 6-16-6 and 7-16-6, the post failed before the post reinforcement reached the yield strain but otherwise the failures were similar to those of the four beams discussed above. The general behaviour of beam 13-16-6-P was similar to that of the first four beams except that as the strut flexure mechanism was forming the top strut failed in shear compression above hole 1 at the reaction end. The failure was explosive and small blocks of concrete from the flange were thrown up to 10 feet. The general behaviour of beams 9-16-7L and 30-21-7 was also similar to that of the first four beams discussed except that in these beams the failure of post 1 caused post 2 to fail and then a strut flexural mechanism was formed over three openings.

The strain in the concrete at the beam centerline indicated the neutral axis was between 3 and 15 inches below the extreme compres-

sion fiber for the last set of readings taken. The concrete strain distributions are similar for beams failing in post shear and for beams failing in flexure for the same applied moments. However, because the flexural capacity was not reached, the maximum compressive strains measured were lower ranging from 0.0010 to 0.0022.

The centerline moment deflection curves for beams failing in post shear, up to their ultimate loads, followed the same path as those of beams failing in flexure for the same loading, flexural reinforcement and clear span. This comparison applies to beams 1-16-6, 3-16-6, 6-16-6, 7-16-6, 13-16-6-P, and 29-12-6 but there are no exact comparisons for beams 4-16-6, 9-16-7L and 30-21-7. Beam 4-16-6 was unique in that the shear capacity of the bottom strut was reduced by the insertion of metal plates and the wrapping of the prestressing strands below holes 1 and 2 of the north shear span. The deflection of this beam was greater than for any other beam for the same applied moment. For this beam, the moment deflection curve was linear to a moment of 396 in-kips after which the slope decreased slowly to a moment of 792 in-kips when post 1 failed. As post 1 failed, there was a jump in the deflection but the beam continued to carry load to a maximum moment of 900 in-kips, 66 percent of the theoretical flexural capacity when the deflection reached 1.86 in. Beam 9-16-7L was different than any of the beams which failed in flexure because it was the only beam with a 7-point loading arrangement and five 7-wire prestressing strands. The centerline deflection for this beam was less than that of beam 8-16-7L which had the same loading and clear span but lighter flexural reinforcement. For beam

30-21-7 there is no direct comparison because there were no flexural failures in the two beams with two point loading and 7 ft shear spans. The centerline deflections of these two beams 30-21-7 and 26-21-7, which had the same loading flexural reinforcement and clear span, were similar for similar load levels. Because of the short pure moment region in these beams the deflection was less than for any other beam of this series. The flexural capacity was not reached in the beams failing in post shear therefore the maximum centerline deflection was less than for beams which failed in flexure. However, the maximum deflection of beam 13-16-6-P was 6.48 inches at 103 percent of the theoretical flexural capacity.

The moment-strain curves for the prestressing strand were similar for beams failing in post shear and beams failing in flexure for the same flexural reinforcement and loading. This direct comparison applies to beams 1-16-6, 3-16-6, 7-16-6, 13-16-6-P and 29-12-6. There were, however, small variations due to differing crack locations but no general pattern was evident. There were no direct comparisons for beams 4-16-6, 19-16-7L and 30-21-7 for the same reasons noted in the discussion of the centerline deflections. In beam 4-16-6, gages 1, 3 and 4 showed that the beam was behaving as an unbonded member for moments above 192 in kips when post 1 failed. For beams 9-16-7L and 30-21-7 there were no exact comparisons except at the beam centerline (gage 1) where the strain was similar to the strain in beams with the same flexural reinforcement for the same applied moments.

Posts 1 and 2 of each of the beams failing in post shear had

the same post reinforcement except in beam 30-21-7 to which supplementary post reinforcement was added in post 1. Four different arrangements of vertical and inclined stirrups were employed in the posts of the beams failing in post shear. Beams 1-16-6, 3-16-6, 4-16-6, 9-16-7L and 30-21-7 had three vertical double legged No. 3 stirrups in posts 1 and 2, beam 30-21-7 also had supplementary post reinforcement in post 1 consisting of three closed No. 3 stirrups set horizontally. Beams 6-16-6 and 7-16-6 had five vertical double legged No. 3 stirrups per post. Beam 29-12-6 had two No. 2 and one No. 3 double legged vertical stirrups in each post. Beam 13-16-6-P had two inclined double legged No. 3 stirrups per post.

The failure of the beams failing in post shear were all initiated in post 1 so the behaviour of the reinforcement in this post is of greater concern. The behaviour will be discussed in numerical order of the gage numbers as they are for other beams.

Gage 5 in post 2 of the beams with vertical stirrups all behaved in a similar manner. The strain remained small until the post cracked at which time there was a jump in the strain reading. After cracking, the strain increased in a linear manner with increasing load to a maximum of less than 56 percent of the yield strain. For beams in which a post crack occurred in post 2, the recorded strains were less in beams with five vertical stirrups per post than those with three for the same loading arrangement. In beams 1-16-6 and 29-12-6, no post cracks developed in post 2 and the maximum strain at gage 5 was less than 1 percent of the yield strain. Post 2 in beam 29-12-6 did not

crack because of the large posts used in this beam. The reason for no crack appearing in post 2 of beam 1-16-6 is not as obvious but since the ultimate load of beam 1-16-6 is only 1.5 kips above the post cracking load for post 2 in beams 3-16-6, 6-16-6 and 7-16-6, it would seem reasonable that it was due to a higher tensile strength in the concrete of the posts of beam 1-16-6. Gage 5 in beam 9-16-7L did not operate correctly as the strain was over 50 percent of the yield strain before the post cracked. Beam 13-16-6-P was the only beam failing in post shear which had inclined stirrups and the behaviour of gage 5 in this beam was different than in beams with vertical stirrups. The measured strain remained small to a load of 8 kips when it increased in a linear manner to the ultimate load yielding at a load of 20 kips. A major crack appeared in post 2 of this beam at a load of 11 kips just above the gage but it did not pass all the way through the post.

For the beams failing in post shear, at failure, post 2 remained intact except for beams 9-16-7L and 30-21-7 in which the failure mechanism extended over three openings. The strain gage readings gave no indication of this type of response.

Post 1 in all of the beams failing in post shear was severely distorted at failure as the failures initiated here. The strains in the post reinforcement for gage 6 were larger in beams failing in post shear than in any other mode. For these reasons the behaviour of the post reinforcement in these beams was of major importance. The shape of the load strain curves for gage 6 were all similar with the strain being small up to the post cracking load when there was a jump in the strain; at

higher loads the strain increased in a linear manner with respect to the applied load. There were, however, differences in the post cracking loads and the slope of the load-strain curves for different reinforcing arrangements, and post and hole geometries. Two different approaches to the correlation of the applied shear and the force in the stirrups are discussed below.

The first was based on the slope of the load strain curve after cracking. Using this slope, the modules of elasticity of the stirrups, the area of the post stirrups, and assuming the strain in all the stirrups is the same, the relationship between the stirrup force and the strain in the post reinforcement was obtained for loads above post cracking.

Beams 1-16-6, 3-16-6 and 4-16-6 had three vertical No. 3 stirrups and 8 by 8 inch vertical posts. The post cracking loads were 9, 8 and 7 kips, respectively, after which the strain increased an average of 142 micor in. per in. per kip shear corresponding to an increase in the axial load in the stirrups of 2.71 kips per kip shear. In beams 6-16-6 and 7-16-6, with five vertical No. 3 stirrups in the 8 by 8 inch vertical posts, the posts cracked at loads of 7 and 6 kips, respectively, after which the strain increased 88 micro in. per in. per kip shear corresponding to an axial load in the five stirrups of 2.80 kips per kip s shear. Beam 9-16-7L had three vertical stirrups in 8 by 8 inches but was loaded with a 7-point load. Post 1 cracked at a load of 2.25 kips giving a maximum shear of 7.88 kips. The slope of the load strain curve beyond post cracking was 531 micro in. per in. per kip load corresponding to an axial load in the three stirrups of 10.2 kips per kip load or in terms

of the total shear 2.90 kips per kip shear. Gage 6 in beam 13-16-6-P with 8 by 8 inclined posts reinforced with two inclined No. 3 stirrups indicated the strain, after the post cracked at a load of 6 kips, increased at a rate of 142 micro in. per in. per kip shear or the axial load on the two stirrups increased at 1.82 kips per kip shear. Beam 29-12-6 had two No. 2 and one No. 3 stirrup in 8 by 12 in. posts. Gages were mounted on the No. 3 stirrup and on one of the No. 2 stirrups and are referred to as gages 6 and 6A, respectively. Post 1 in this beam cracked at a load of 11 kips. Gage 6 after the post cracked indicated the strain was increasing at a rate of 275 micro in. per in. per kip shear or the axial load in the stirrups was increasing at 3.35 kips per kip shear while gage 6A indicated the axial load in the stirrups was increasing at 1.97 kips per kip shear. The 8 by 8 in. posts of beam 30-21-7 were reinforced with three vertical and three horizontal stirrups. The holes of this beam were 21 inches long whereas those of the other beams failing in post shear were 16 inches long. After post 1 cracked at a load of 6 kips, the strain increased 129 micro in per in per kip shear corresponding to an axial load in the three vertical stirrups of 2.47 kips per kip shear. Gage 41 on the horizontal stirrups indicated the strain was increasing at 106 micro in. per in. per kip shear or 2.03 kips per kip shear axial load in the three horizontal stirrups.

This first approach is conservative because an extension of the load strain curve after cracking to the abscissa indicates a compressive strain exists in the reinforcement at zero load. The second approach is also conservative but passes through 0 at 0 load.

The second approach is based, as was the first, on the assumption that the strain in all of the stirrups is equal; but instead of taking the average slope after the formation of a post crack, the slope was calculated from zero to a point on the load strain curve where the yield strain was reached or the last available point on the curve for beams where no yielding occurred. Using the slope of this line, the area of the reinforcement and the elastic modulus, the relationship between the total shear and the axial load in the reinforcement was obtained. This calculation showed that for a given geometry the load on the post reinforcement was a multiple of the applied shear. For 8 by 8 inch posts and 8 by 16 inch rectangular holes with vertical reinforcement the axial load on the post reinforcement was 2.51 times the beam shear with a maximum variation of 6.3 percent for the six beams failing in post shear in this category. Each of the other three beams failing in post shear had different geometries. In beam 13-16-6-P with inclined openings and stirrups the axial load in the stirrups was 1.83 times the applied shear. Gages 6 and 6A in beam 29-12-6 indicated the force in the post stirrups was 1.55 and 1.18 kips per kip shear, respectively. Beam 30-21-7 with 21 inch holes had posts which were reinforced by both horizontal and vertical stirrups indicated the axial load on the stirrups was 2.42 of the beam shear while gage 41 indicated the horizontal stirrups carried a load of 1.97 times the applied shear.

The principal force acting on the posts in the shear spans was the horizontal shear due to the change in moments between the centerlines of the holes. Therefore, the maximum force in the stirrups calculated by

the second method were compared to this horizontal shear for all of the beams of this series with more than one hole in the shear span for various post and reinforcement arrangements. Where both vertical and horizontal stirrups were used, the maximum average force in the vertical stirrups for hole spacings of 24 and 29 inches were 1.24 and 1.22 times the horizontal shear force, respectively; in the horizontal stirrups the force was 0.78 and 0.99 times the horizontal shear, respectively. When only vertical stirrups were placed in the vertical posts for holes 24 inches on center, the post size had a large effect on the force in the stirrups. For 8 by 8 posts, the maximum average force in the stirrups was 1.56 times the horizontal shear while for 8 by 12 in. posts the maximum average force in the stirrups was only 1.03 times the horizontal shear. For inclined stirrups, the post geometry also affected the force in the stirrups. In beams with parallelogram shaped openings 16 inches long spaced at 24 inches on center, the maximum average force in the stirrups was 1.10 times the horizontal shear while for beams with 8 by 12 in. posts and holes spaced at 24 inches on center the force in the inclined stirrups was only 0.75 times the horizontal shear. For beams with 7-point loads and 8 by 16 in. holes 24 inches on center, the maximum average force in the vertical stirrups was 1.87 times the horizontal shear. This comparison of the stirrup force and the horizontal shear shows the advantages of (i) adding horizontal stirrups to vertical stirrups, (ii) inclining the reinforcement, and (iii) increasing the post size.

There were no gages placed on the post reinforcement in the post flexural tension zone. However, from the cracking patterns, the

post flexural capacity appeared to be adequate for the post and reinforcement arrangements used in this series.

At gage 7 in the solid shear span, the load strain plots for beams failing in post shear were similar to the curves for gage 7 of the beams failing in flexure with 6 foot shear spans with two exceptions; beams 4-16-6 and 29-12-6. Generally, the strain was small up to a load of 4 to 7 kips after which the strain increased linearly towards yield between 18 and 25 kips but only in beam 13-16-6-P did gage 7 indicate that the yield strain was reached. In beams 4-16-6 and 29-12-6 the recorded strains at gage 7 were less than for other beams of this group. Beam 4-16-6, as was noted earlier, was cast with the shear capacity of the bottom strut reduced and, because of this, gage 7 indicated a small compressive strain up to the ultimate load. Gage 7 in beam 29-12-6 for no apparent reason behaved similarly to gage 7 in beams with 4 foot shear spans. The strain remained small to a load of 11 kips after which it increased slowly to a maximum strain of 750 micro in. per in. at a load of 19 kips.

The top struts of the beams failing in post shear had a variety of top strut shear reinforcing arrangements consisting of vertical or inclined No. 2 stirrups. Beams 1-16-6, 3-16-6, 4-16-6, 6-16-6, 7-16-6 and 9-16-7L had vertical stirrups spaced at 3.50 inches. Beams 29-12-6 and 30-21-7 had vertical stirrups at 3.0 and 2.875 in., respectively, while 13-16-6-P had inclined No. 2 stirrups spaced at 3.50 inches. The recorded strains were highest above hole 2 for all but beams 7-16-6 and 9-16-7L of this group and along the length of the

strut in the stirrup closest to the load for gages 9 and 13 for struts above holes 2 and 1, respectively. Beam 6-16-6 was the only beam with vertical stirrups in which the gages indicated yielding. Gage 9 in this beam reached the yield strain at a load of 15.5 kips while gage 13 reached 90 percent of the yield strain at the ultimate load of 20.5 kips. For all other beams the strain for gages 9 and 13 was less than 41 percent of the yield strain. The recorded strains at gage 9 in beam 4-16-6 were similar to those of beam 6-16-6 but beam 4-16-6 failed at a much lower load. In beam 13-16-6-P, with inclined stirrups, the maximum recorded strains were larger than those of all but beam 6-16-6, the ultimate load was also larger. At gage 9 in beam 13-16-6-P the strain increased slowly to 350 micro in. per in. at a load of 18 kips; at higher loads the strain increased rapidly to the yield strain at a load of 22 kips. The maximum strain above hole 2 was more than three times that above hole 1.

The shear reinforcement in the bottom struts of the beams failing in post shear consisted of closed No. 2 stirrups. Beams 1-16-6, 3-16-6, 6-16-6, 7-16-6, 9-16-7L and 13-16-6-P had the same stirrup spacing and inclination in the bottom strut as in the top strut. Beams 29-12-6 and 30-21-7 had vertical No. 2 stirrups spaced at 1.625 and 1.875 inches, respectively. Beam 4-16-6 had no shear reinforcement in the bottom strut. The strain in the stirrup of the bottom strut was larger below hole 2 except for beams 6-16-6 and 29-12-6 where the stirrup strain was equal below both holes of the shear span. The maximum strain along the length of the bottom strut was recorded near the strut center-

line. However, only in beams 13-16-6-P, 29-12-6 and 30-21-7 were gages mounted at the strut centerline at gage locations 19 and 23; for the other beams the gages closest to the centerline were gages 18 and 22. Of the beams with vertical stirrups, only in beam 30-21-7 was the yield strain reached while in all of the other beams the strain was less than 40 percent of the yield strain. The gages on the inclined stirrups of beam 13-16-6-P indicated the strain was larger than in the beams with vertical stirrups. The strain in the bottom strut stirrups of beam 13-16-6-P below hole 2 was twice as large as the strain below hole 1.

The strut flexural reinforcement in the beams failing in post shear had various arrangements. In beams 1-16-6, 3-16-6, 6-16-6, 7-16-6, 9-16-7L and 13-16-6-P the top strut flexural reinforcement consisted of six No. 3 bars, four at the top and two at the bottom. The bottom strut had two No. 3 bars as supplemental longitudinal reinforcement in the bottom of the strut only. Beam 4-16-6 had the same top strut flexural reinforcement as the beams discussed above but the bottom strut had no supplementary flexural reinforcement. Beam 29-12-6 had six No. 2 bars in the top strut and four No. 2 bars in the bottom strut, two at the top and two at the bottom. Beam 30-21-7 had four No. 3 at the top of the top strut and two No. 4 bars at the bottom. In the bottom strut of beam 30-21-7, the supplemental longitudinal reinforcement consisted of four No. 4 bars, two at the top and two at the bottom. Beams 29-12-6 and 30-21-7 were the only beams with gages on the strut flexural reinforcement.

The general behavior of the reinforcement as indicated by the

gages on the supplementary longitudinal reinforcement in these beams was similar to the behaviour of the reinforcement in beams with 6 foot shear spans which failed in flexure. Vierendeel strut action was indicated over each opening but there were also indications of the formation of a mechanism over two openings and some decrease in stress possibly due to loss of bond. The smaller holes and lighter reinforcement of beam 29-12-6 and the larger holes and heavier reinforcement of beam 30-21-7 produced very similar effects on the load strain curves for these two beams. The first yielding of the strut flexure reinforcement occurred below hole 2 at the bottom of the bottom strut near the load at gage 34 where there is tension due to both beam and strut moments. The maximum compressive strains were recorded in the bottom of the top strut above hole 1 near the reaction at gage 32.

The posts in the shear spans are subjected to a horizontal shear which places high demands on the post reinforcement. However, when the post reinforcement is adequate, the failure is forced to another location.

5.3.3 Beams Failing in Strut Shear

Strut shear failure is the failure of a beam due to a shear compression failure of the top strut and/or a shear and tension failure of the bottom strut. This mode of failue is undesirable in that it can occur with little warning below the flexural capacity of the beam. Strut shear failure was the most common failure mode in the test series of LeBlanc (6) and Sauve (12). However, the addition of strut shear

reinforcement in the top and bottom struts of the beams in this series eliminated this type of failure from all but 5 beams, namely, 2-16-4, 5-11-4, 11-16-4, 15-12-6, and 16-12-6.

The top and bottom struts are acted on by shear, moment and axial loads. The total shear at a given section through a hole is distributed in some changing ratio between the top and bottom struts. This strut shear also produces strut moments. The axial load on the struts due to the beam moments produces tension in the bottom strut and compression in the top strut.

The axial load on the strut can be accurately calculated from the bending moment at the centerline of the hole and from the assumption that the compressive force acts through the centroid of the top strut and the tensile force acts through the centroid of the tensile reinforcement in the bottom strut. The shear and bending moments in the struts are, however, dependent on the distribution of shear to the top and bottom struts which changes as the load is applied. Several different approaches to the distribution of this shear have been presented. The simplified classical Vierendeel truss analysis assumes the shear is distributed in proportion to the gross concrete areas of the struts and contraflexure points are assumed at the center of the strut's length. Lorensten (7) in his work assumed the shear was carried totally by the top strut while the bottom strut was assumed to be a tension link carrying no shear. Nasser et al (8) suggested a design procedure which proportioned shear by gross concrete area but suggested any distribution can be assumed as long as the total shear is provided for.

Hanson (5), from his research on joists in negative bending, concluded that before cracking, the shear was distributed in proportion to concrete area and after cracking the compressive strut carried all additional shear. At this time, it is recognized that the beam shear is resisted by shear in both the top and bottom struts. The relationship between the strut forces and the parameters of this series is discussed in the following paragraph.

The reinforcement in the region around the holes was the prime variable in this test series. The shear reinforcement in the struts had the greatest influence on the struts' ability to resist shear. Decreasing the stirrup spacing increased the strut shear failure load. The stirrup spacings for the first 16 beams of this series were set at 3 1/2 inches in both top and bottom struts based on calculations of the net section through a hole. The spacings of the stirrups in the top and bottom struts of beams 17 to 30 were based on an assumed shear distribution according to "shear area". The "shear area" of the bottom strut was calculated using the minimum web width. The "shear area" of the top strut was based either on the minimum web width or the minimum web width plus a portion of the flange and chamfers within the flange thickness on either side of the web. No strut shear failures occurred in the beams where the shear was proportioned by "shear area".

Strut flexural reinforcement played a major role in resisting the strut moments due to strut shear and probably carried some shear by dowel action. It did not appear to have a great effect on the strut shear capacity of beams of this series.

The post reinforcement and its arrangement had a large influence on the strut shear capacity. Inclining the post reinforcement in otherwise similar beams increased the shear capacity of the top strut near the load sufficiently to cause a flexural failure. Other post reinforcement arrangements and the reinforcement arrangements in the solid shear span had little effect on the strut shear capacity.

The various 2-point and the 7-point loadings effectively changed the total applied shear to produce the same bending moment. The 2-point loading with 4 foot shear spans gave the highest shear which placed the greatest demands on the struts of hole 1 in shear flexure and axial load. Three of the 8 beams with this loading failed in strut shear, namely, beams 2-16-4, 5-16-4, and 11-16-4-P. The 2-point loading with 6 foot shear spans was the other loading condition which produced strut shear failures. This loading produced the highest axial loads in the struts of hole 2 and thereby increased the shear carrying capacity of the top strut and also decreased the capacity of the bottom strut. At hole 1, because the axial load was smaller, the bottom strut was able to carry more shear than the bottom strut of hole 2, while the top strut at hole 1 had a lower capacity. For these reasons and because the depth of the bottom strut was 1.66 times the depth of the top strut, the critical struts should be above and below hole 2. This was the case in the two beams of this group with 6 foot shear spans which failed in strut shear, namely, beams 15-12-6 and 16-12-6. The loadings with 5 and 7 foot shear spans produced conditions similar to those of 4 and 6 foot shear spans, respectively, but were applied to beams with longer

holes. The 7-point loading arrangement gave a large variation in shear and moment along the length of the beam but the total applied shear was less than for the 2-point loads with shear spans less than 6.5 feet.

Increasing the flexural capacity by increasing the number of prestressing strands had two primary effects on the struts. The total shear required to produce flexural failure was increased placing extra demands on the shear reinforcement. Secondly, the effective axial tension in the bottom strut was reduced because of the higher initial prestress force thus increasing the shear capacity of the bottom strut.

The geometry of the holes and posts had some influence on the shear capacity of the struts but the most evident changes were in association with post reinforcement arrangements which were discussed earlier. One geometrical property which was not related to post reinforcement was the hole length. For otherwise similar beams, the strut shear capacity was higher as the hole length decreased (compare 15-12-6 and EL5-16-6 (6) with ultimate loads of 19 and 17.25 kips, respectively).

The strut shear failures of this series, as shown by the photographs of failure, occurred in two different ways: (i) by the simultaneous opening of shear cracks in both the top and bottom struts, and (ii) by opening of shear cracks in the top strut and the formation of a mechanism in the bottom strut. Beams 2-16-4, 15-12-6 and 16-12-6 failed in the former manner while beams 5-16-4 and 11-16-4-P failed in the latter way.

Beam 2-16-4 failed when the shear reached 24.5 kips at 85

percent of the flexural capacity. Failure occurred as existing cracks in the top and bottom struts opened. In the top strut, the critical crack was a web shear crack which started near the middle of the strut and sloped toward the load at an angle less than 45°. The yield strain was reached in the strut shear reinforcement before failure. The failure crack in the bottom strut was a flexure-shear crack at approximately 45°. No yielding of the bottom strut stirrups was recorded prior to failure.

Beams 15-12-6 and 16-12-6 failed at shears of 19 kips and 24 kips, respectively, at 81 and 103 percent of theoretical flexural capacity. These two beams were identical except that beam 15-12-6 had no strut shear reinforcement while beam 16-12-5 had No. 2 stirrups spaced at 4.0 inches. Both of these beams had two holes in each shear span and the failures were through the struts of hole 2. The critical shear cracks in both the top and bottom struts were in the same locations on the critical struts as those of beam 2-16-4. Beam 15-12-6 had no strut shear reinforcement, therefore, it failed at the load at which the web shear crack appeared. Beam 16-12-6 did have strut shear reinforcement which picked up load quickly after the formation of the web shear crack, and increased the beam's capacity to beyond the theoretical flexural capacity. As the shear failure occurred, there was a violent release of energy which led to a web flange separation and a compression failure of the top strut in the pure moment region 8 feet from the shear failure.

Beams 5-16-4 and 11-16-4-P which formed the second set had

the same critical web shear crack in the top strut as the other beam but the bottom struts failed in different ways. Beam 5-16-4 failed when the shear was 20.5 kips or at 72 percent of the theoretical flexural capacity. Both legs of the center stirrup of the top strut were ruptured after failure. The bottom strut of this beam had its shear capacity reduced by the insertion of four metal plates in the strut but as seen from the cracking pattern, some shear was carried by dowel action and friction across the metal plates. The final shape of the bottom strut suggests a rotation at both ends similar to a strut flexure mechanism. Beam 11-16-4-P failed when the shear reached 29 kips or 78 percent of the theoretical flexural capacity. This beam had parallelogram shaped openings and sloped stirrups. The failure of the top strut was similar to the failures of the other beams though the strain in the stirrups was slightly less for the same load. The critical web shear crack appeared in the top strut at a load of 24 kips and opened considerably at failure. However, the bottom strut did not fail in shear but a hinge formed at the load end about which the beam rotated causing concrete crushing at the top and opening a large crack along the post reinforcement.

The moment deflection curves for beams failing in strut shear were much like those of similar beams failing in flexure up to the ultimate loads of those failing in strut shear. For beams 2-16-4, 5-16-4 and 11-16-4-P, there were no similar beams because of differences in span length and flexural reinforcement. The deflection of beams 2-16-4 and 5-16-4 was larger than those of the beams which failed in

flexure because these two beams had less flexural reinforcement and longer spans. Beam 11-16-4-P also had larger deflection because of the longer span. These three beams all had very similar moment-deflection curves up to a moment of 720 in-kips with the deflection of 5-16-4 slightly larger because of the reduced shear capacity of the bottom strut. At moments above 720 in-kips the slope, though not as steep, was constant to the maximum deflection for beams 2-16-4 and 5-16-4 of 2.4 inches and 1.79 inches, respectively. The slope of the moment-deflection curve for beam 11-16-4-P remained small to a moment of 864 in-kips above which the slope though not as steep was constant to a maximum deflection of 2.32 inches.

The major change in slope for the three beams above occurred at the load at which the flexural cracks completely cut the bottom strut in the pure moment region. The final flexural crack had little effect on the beam response.

For beams 15-12-6 and 16-12-6, there are exact comparisons as beam 12-16-6-P and 14-12-6 had the same loading (6 foot shear spans), and the same clear span (20 feet) and flexural reinforcement (5 strands). The moment-deflection curves for these beams were almost identical. The slopes are linear up to a moment of 720 in-kips when flexural cracking occurred and increased slowly to a moment of 936 in-kips when the cracks extended completely through the bottom struts in the pure moment region. Above 936 in-kips the moment-deflection curve was relatively linear to a moment of 1296 in-kips as the cracks indicated the neutral axis was within the top strut. The maximum deflection of beam 15-12-6

was 2.16 inches at 81 percent of the theoretical ultimate flexural capacity while the deflection of 16-12-6 reached 7.76 inches at 103 percent of the theoretical flexural capacity, only slightly less than the maximum deflection of beams which failed in flexure.

The moment strain diagrams for the prestressing steel had the same shape as the moment-deflection curves. Gages at location 1 and 2 in the pure moment region for all the beams failing in strut shear and flexure with similar flexural reinforcement indicated similar strains for the same applied moment. The moment strain diagram for gage 2 in beam 2-16-4 was the only exception because the supplementary reinforcement in this beam continued 2 feet beyond gage 2 whereas in all the other beams of this series the strut flexural reinforcement was terminated at gage 2, that is, below the first hole in the pure moment regions. The gages in the shear spans were less consistent than those in the pure moment region due most likely to differences in cracking patterns. For beams with 4 foot shear spans (2-16-4, 5-16-4 and 11-16-4-P), the strains at gage 4 in the shear span were greater than those of similar beams which failed in flexure because the areas of the supplemental longitudinal reinforcement in the beams failing in flexure were larger. For the beams failing in strut shear, the strains at gage 4 for some stages of loading were greater than the strain in the pure moment region. For beams failing in strut shear with 6 foot shear spans, gages 3 and 4 indicated less strain than those in the pure moment region for the same moment but more than for similar gages in beams with similar flexural reinforcement

and loading which failed in flexure.

The long stirrups in the posts and solid shear spans of the beams failing in strut shear behaved in a manner similar to those of beams which failed in flexure. The post and solid shear of all the beams which failed in strut shear were reinforced with double-legged No. 3 stirrups set vertically except in beam 11-16-4-P where they were inclined at 45°. Three stirrups were placed in the posts in the shear span and below the loads. Gages were mounted at gage location 5 only in beams 15-12-6 and 16-12-6 of this group. Because of the large post size and because the load was above post 2, the recorded strains at gage 5 for these two beams were less than 2 percent of the yield strain. All of the beams of this group had gages mounted at gage location 6 in post 1. This gage was below the load for the beams with 4 foot shear spans. In beams 2-16-4 and 5-16-4, the strain was small up to the post cracking load where there was a jump in the strain; after the post cracked, the strain increased slowly to a maximum strain less than 66 percent of the yield strain. There was no post crack in post 1 of beam 11-16-4-P passing through the gage. Therefore, the maximum strain was small reaching only 34 percent of the yield strain. Because beams 15-12-6 and 16-12-6 had 6 foot shear spans, post 1 and gage 6 were within the shear span. The load strain plots for these beams indicated a small strain to post cracking when there was a jump in the strain after which the strain increased to failure. The load-strain curves in beam 16-12-6 were more irregular than those of beam 15-12-6 as there were two jumps; one as the post cracked below the

the gage at a load of 11 kips, and the second as a crack passed above the gage at 24 kips. Calculation of the average force in the stirrups to the last point on the load strain diagram (88 percent of the yield strain), indicates the force in the stirrups was 1.71 and 1.34 times the applied shear for beams 15-12-6 and 16-12-6, respectively. The force in the post stirrup of beam 29-12-6 which had the same post and hole size, and which failed in post shear, was 1.55 times the applied shear force.

In the solid shear spans the stirrups were spaced at 8 inches for all five beams of this group. Gage 7 mounted on the first stirrup of the solid shear span indicated the yield strain was reached in three of the beams (2-16-4, 11-16-4-P, and 16-12-6). In all cases, the stirrups were adequate as indicated by the cracking patterns which showed that in no case did the shear cracks in the solid shear span cross more than two stirrups or extend to the bottom of the beam. The strain in beams 5-16-4 and 15-12-6 remained below 75 percent of the yield strain. The strain in beam 5-16-4 was irregular because of the reduced shear capacity of the bottom strut. In beam 15-12-6, the strain was less than in most of the other beams with 6 foot shear spans.

The shear reinforcement provided in the struts of the beams of this group was not sufficient to prevent strut shear failures. The strut shear reinforcement of beams 2-16-4, 11-16-4-P and 16-12-6 consisted of closed No. 2 stirrups in both the top and bottom struts of the shear spans with spacings of 3.50, 3.50 and 4.0 inches, respectively. The reinforcement was set vertically except in beam 11-16-4-P

where it was inclined at 45°. Beam 5-16-4 which had the shear capacity of the bottom strut reduced had closed vertical No. 2 stirrups in the top strut only spaced at 3.50 inches. Beam 15-12-6 had no strut shear reinforcement.

In the top struts, some of the gages in all of the beams indicated yielding before strut failure occurred. For the beams with vertical stirrups, the strains were highest in the second stirrup from the load end of the strut (gages 10 and 14) and for the hole with the highest axial loads on the struts. For beam 11-16-4-P with inclined stirrups, the stirrup closest to the load end of the strut (gage 9) was the first to indicate yielding. The behaviour of these critical gages was similar for both loadings, being small up to the load at which a web shear crack appeared and increasing rapidly to yield before failure. The rapid increase in strain after cracking indicated the inadequacy of the strut shear reinforcement of these beams. The load-strain curves for beam 11-16-4-P with inclined stirrup indicated less strain for a given load than the curves of the vertical stirrups. This indicated their greater ability to resist shear. In the struts near the failure, the stirrups behaved in a similar manner for both loadings. However, this was not the case for similar locations with different loading conditions. The strains above hole 1 for beam 16-12-6 were much less than the strains in the stirrups above hole 1 for the beams with 4 foot shear spans. Beam 15-12-6 with no strut shear reinforcement was identical to beams 16-12-6 and 14-12-6 except that 16-12-6 had strut shear reinforcement and 14-12-6 had inclined post reinforce-

ment. Beam 15-12-6 failed at 19 kips which is the same load at which a web shear crack appeared in beam 16-12-6. However, beam 16-12-6 continued to carry load to 24 kips showing the value of strut shear reinforcement. Beam 14-12-6 which failed in flexure had no strut shear reinforcement either but, because the sloped stirrups in the posts arrested the shear cracks, this beam carried a maximum shear of 25 kips before failing.

Only three beams of this group had bottom strut shear reinforcement. The gages on these stirrups indicated that there was little difference in the behaviour of the reinforcement due to different loading and a small difference due to inclination. The general behaviour for beams with vertical stirrups was nearly the same for both 4 and 6 foot shear spans. The strain increased in compression to about 11 kips after which the strain increased in tension to failure. In no case was the yield strain reached, indicating a reserve shear capacity in the bottom strut. In beams with 6 foot shear spans, the strains were 23 percent higher at hole 1 than at hole 2 yet the failure occurred through hole 2. The gages on the inclined stirrups of beam 11-16-4-P indicated the strain was greatest in the center stirrup at gage 23. This strain was twice as large as the maximum recorded strains at gage locations 21 and 22. The shape of the load strain curves for all the bottom strut stirrups of beam 11-16-4-P had similar shapes increasing in tension slowly to a load of about 10 kips after which the strain increased more rapidly to failure.

From the strain measurements taken on the strut shear rein-

forcement for all 30 beams tested some general observation can be made. The strut stirrups with the highest strains were generally those above and below the hole where the strut axial loads were highest, that is, where the beam moment was the highest. The gage along the length of the strut with the highest strain was usually the nearest to the load in the top strut and near the strut centerline in the bottom strut. The location of the failure cannot be absolutely determined, but it is clear that the strut shear reinforcement prevented the strut shear failure of over 80 percent of the beams of this series. Further, none of the beams with their shear proportioned by shear areas and strut shear reinforcement designed by section 11.4.3 of the ACI Code (1) considering strut axial loads due to beam moments and proportioning the shear by shear areas failed in strut shear. Though in some cases the maximum spacing and the maximum allowable shear stress specified in section 11.6.4 to prevent concrete crushing were exceeded, no concrete crushing was noted along the shear cracks.

There were no gages on the strut flexural reinforcement of the beams which failed in strut shear. Strut flexure cracking is evident in the photographs but the strut flexure reinforcement was adequate to resist the applied loads.

5.3.4 Beams Failing in Strut Flexure

Strut flexural failure ideally is a failure in which both ends of both struts above and below a hole fail in flexure yielding the reinforcement and crushing the concrete. This forms a strut flexure

mechanism over one opening with hinges at both ends of the struts. This definition is rather ideal and assumes sufficient rotational capacity is available for the formation of all four hinges. However, in some cases, after the formation of one hinge there is a sufficient change in the concrete sections to cause failure before all four of the hinges have developed.

Strut flexural failure is generally undesireable in that the flexural capacity of the beam is not reached. There was, though, visible warning of this type of failure in the spalling of the concrete as the hinges were forming. Strut flexural failures are generally associated with long holes. However, in beams with shorter holes, the anchorage of the strut flexural reinforcement becomes more difficult.

The forces acting on the struts of the shear spans which lead to strut flexural failures are shear axial load and strut moments. The critical factors affecting these forces and the strut flexural capacity of a beam are the distribution of the shear to the top and bottom struts and the location of inflection points along the strut length. The distribution of the shear was discussed in the previous section (section 5.3.3). Several approaches have been used to locate the inflection points in the struts but the conclusion was the same: for design it is sufficiently accurate to assure the inflection point at the strut centerline as is done in the simplified Vierendeel truss analysis. Nasser et al (8) calculated the location of the inflection points from the deflected shape of the struts and from concrete strains at the quarter points of the strut. They found the point of inflection

could be 0.10 of the hole length on either side of the strut centerline for their long symmetrically reinforced struts (1.3 to 2.6 times the beam depth). Hanson (5) measured the concrete strains along the length of the compressive struts of his joists in negative bending and concluded the inflection point was at the strut centerline. Ramey and Tattershall (10) using a finite element analysis found that the point of inflection was within 0.10 of the hole length to the strut centerline for beams having holes with a height of 0.50, the height of the beam, and located at the centroid. All of these tests suggested the simplified Vierendeel truss analysis with its assumption of inflection points at the strut centerline is adequate.

The shear carried by each strut acting at the inflection points produce the strut moments. These strut moments act in the same direction at either end of the struts. Therefore, the stresses at any given level change from tension to compression in the length of the strut or across a post in the shear span. Because of this, the bond stresses achoring the strut flexural reinforcement are high.

The relationships among the strut forces, the beam parameters, and the strut flexural capacity indicated that the strut flexural reinforcement and the hole length were the major factors with the other parameters having a much smaller influence on the strut flexural behaviour.

The reinforcement in the region around the holes was the prime variable of this series. The strut flexural reinforcement placed

in the struts of the shear spans had a great influence in the strut flexural capacity. Increasing the area of the strut flexural reinforcement increased the capacity of the beam. This was evident in studying the failure loads and reinforcement of beams which failed in strut flexure and other modes. The failure loads of beams 20-26-5 and 22-26-5 with 1.10 in^2 of supplemental longitudinal reinforcement was 22 kips, while beam 21-26-5 with 2.30 in^2 of supplemental longitudinal reinforcement failed at a load of 29 kips. All three of these beams failed in strut flexure. This comparison is also true for beams 25-16-6 and 17-16-4 which failed in S.F. with lighter reinforcement than beams 19-16-6 and 18-16-4, respectively, which failed in flexure. There was only one case in which increasing the area of the strut flexural reinforcement did not increase the beam capacity. Beam 26-21-7 had lighter strut flexural reinforcement but failed at a higher load than beam 30-21-7. However, beam 30-21-7 also had lighter post reinforcement and failed in post shear. Increasing the area of the strut flexural reinforcement by increasing the bar size increased the bond stresses and the possibility of slip in the reinforcement.

While the strut shear reinforcement was adequate to prevent strut shear failure, it had little influence on the strut flexural hehaviour. Decreasing the spacing did not increase the capacity. The presence of stirrups in excess of those required to carry the shear should, however, prevent the longitudinal splitting associated with bond failure.

The post reinforcement had little effect on the strut flexural

behaviour as long as it was sufficient to prevent post shear failure which drastically increases the effective hole length causing a strut flexural failure over two openings.

The shear reinforcement in the solid shear spans also had to be sufficient to prevent elongation of the hole by splitting along the strut flexural reinforcement.

Beam geometry including hole length and shape and post length affected the strut flexural behaviour. Hole or strut length was the most important geometric parameter. As the hole length increased, the strut moments increased. This was emphasized by the observation that no strut flexural failures occurred in the beams with the shortest holes (12 inches long), and all of the beams with the longest holes (26 inches long), had strut flexural failures. The shape of the opening influenced the strut length or at least the effective strut length. For rectangular holes with rounded corners of the proportions used in this series, the effective strut length was the maximum horizontal longitudinal dimension of the hole. For openings shaped differently, such as parallelogram or circular, the effective hole length was not the maximum horizontal longitudinal dimension but something less. The effective hole length for various shaped openings was not a major factor in this test series. However, in Redwood's paper "Design of Steel Beams with Web Openings" (11) some solutions to this problem are discussed for steel beams.

The post size had only a limited effect on the strut flexural behaviour. Larger posts reduced the tendency to form a strut flexural

mechanism over two openings and provided a greater length for the anchorage of the strut flexural reinforcement.

The loading conditions were varied to place different demands on the reinforcement in the region around the holes in the shear spans. Strut flexural failures occurred in beams with four of the five different loading arrangements in this series.

Higher flexural capacity, like shorter shear spans, produced greater demands on the reinforcement in the shear spans. Increasing the flexural capacity, as was done in this series by increasing the number of prestressing strands, also increased the strut flexural capacity of the bottom strut. This capacity was increased in two ways: (i) by reducing the tension in the concrete and the mild steel reinforcement of the bottom strut and (ii) by increasing the area of strut flexural reinforcement in the bottom strut.

The failures of all of the beams failing in strut flexure were characterized by two occurrences: (i) yielding of the reinforcement, and (ii) crushing of the concrete at the bottom of the top strut near the reaction end, above the hole with the highest axial loads. As discussed below, these two events occurred prior to failure in each of the beams.

Beams 20-26-5 and 22-26-5 had ideal strut flexural failures with hinges forming at all four locations. The first hinge formed at a load of 19 kips with concrete crushing at the bottom of the top strut near the reaction of both beams. At a load of 22 kips or 79

percent of the theoretical flexural capacity failure occurred as the strut flexural mechanisms were completed in both beams.

Beam 21-26-5 with the same geometry but much heavier strut flexural reinforcement than beam 20-26-5 and 22-26-5 failed at an applied shear of 29 kips or 104 percent of the theoretical flexural capacity. For this beam, the first signs of the formation of a strut flexural mechanism occurred at a load of 26 kips as concrete spalling was observed at the bottom of both the top and bottom struts near the reaction. At the failure load, shear cracks opened in the top strut near the load and the strut buckled downwards.

In beams 17-16-4 and 25-16-6, the concrete spalling from the formation of the first hinge caused a sufficient reduction in the concrete suction resisting shear that a large shear crack opened causing failure. The ultimate load on beam 17-16-4 was 31 kips or 89 percent of the theoretical flexural capacity. For beam 25-16-6, the ultimate load was 22 kips or 94 percent of the theoretical flexural capacity.

Beam 26-21-7 failed at a load of 20 kips or 100 percent of the theoretical flexural capacity. The first concrete crushing was noted at a load of 19 kips as a hinge developed at the reaction end of the top strut above hole 2. After the failure load had been applied for several minutes, the load started dropping off as two more hinges formed in the top strut of hole 2. The load was then removed and pictures were taken. Upon reloading to 19 kips, concrete crushing at

the load end of the top strut continued unchecked and the beam could no longer carry load.

The concrete strains at the beam centerline for the beams failing in strut flexure were similar to those of beams failing in flexure and indicated that the strain distribution was reasonably linear throughout the depth of the section. Beams 21-26-5 and 26-21-7, which reached their theoretical flexural capacity, had the highest recorded compressive strains in the concrete at over 0.0030 in. per in. and the strain distribution indicated the neutral axis was within the top strut. In the other beams in which failure occurred below the theoretical flexural capacity, the maximum recorded strain in the concrete was between 0.0011 and 0.0016 and the strain distribution indicated the neutral axis was between 3 and 12 inches from the top of the beam.

The moment-deflection curves for the beams which failed in strut flexure are very similar to those of beams failing in flexure with the same span length. The largest differences were found in beams 20-26-5 and 22-26-5 which had the longest holes and relatively light strut flexural reinforcement.

The moment-deflection curve for beam 17-16-4 follows the path of 18-16-4, 23-16-4, 27-16-4 and 28-16-4 which all failed in flexure and had heavier strut flexural reinforcement up to a moment of 1344 in-kips at which time concrete spalling was observed in the top strut of beam 17-16-4. Above this load, the deflections increased more

rapidly than in the beams failing in flexure, up to the ultimate load at which time the deflection was 2.75 inches. For beams 20-26-5, 21-26-5 and 22-26-5 there was no direct comparison because all of the beams with 5 foot shear spans failed in strut flexure. The deflections of these beams were slightly larger than those of beams failing in flexure with both 4 and 6 foot shear spans. The moment deflection curves for these beams were linear to a moment about 600 in-kips at which time there were flexural cracks in the pure moment region of each of the beams. The slope of the curves above this load decreased very slowly to a moment of 900 in-kips at which time flexural cracks passed completely through the bottom struts. Above 900 in-kips the slope decreased more rapidly and beams 20-26-5 and 22-26-5 with lighter strut flexural reinforcement appeared to be less stiff than beam 21-26-5. For beams 20-26-5 and 22-26-5 above a moment of 900 in-kips, the deflection increased rapidly to a maximum of 2.13 and 2.21 inches, respectively. For beam 21-26-5, the deflection increased less rapidly and followed the moment deflection curves of beams with 4 and 6 foot shear spans which failed in flexure up to the ultimate load when the deflection reached 5.19 inches. The moment deflection curve for beam 25-16-6 followed the same path as beams 19-16-6 and 24-16-6 which failed in flexure and had the same flexural reinforcement in the pure moment region and span length though the strut flexural reinforcement in the shear spans was lighter in beam 25-16-6. There was no direct comparison for beams 26-21-7 because none of the beams with the same loading failed in flexure. The moment deflection curve for this beam was, however, very similar to those of beams 19-16-6 and 24-16-6 which

failed in flexure. The deflections of beam 26-21-7 at moments below 1400 in-kips were larger, and above 1400 in-kips were smaller than those of the beams which failed in flexure. The slope of the moment deflection curve was linear to a moment of 672 in-kips when flexural cracks appeared just outside the pure moment region. The slope at higher loads increased slowly to a moment of 1512 in-kips after which the deflections increased more rapidly to failure reaching 3.06 inches.

The moment-strain curves for the prestressing strand at various locations along the length of the beam all had the same shape as the moment deflection curves. In the pure moment region at gage 1 the behaviour of all of the beams failing in strut flexure and all other beams with five prestressing strands were very similar, being linear up to the flexural cracking load with the slope at higher loads decreasing to failure. In the shear spans at gage locations 3 and 4, the strain was less than in the pure moment region and varied with the number of holes in the shear span and the area of the supplemental longitudinal reinforcement. Beams 25-16-6 and 26-21-7 had two openings in each shear span. Though the strains in both these beams decreased towards the supports, the strains in beam 25-16-6 were much larger because of the lighter supplemental longitudinal reinforcement. Beams 17-16-4, 20-26-5, 21-26-5, and 22-26-5 all had one hole in the shear spans. Gage 4 at the centerline of this hole indicated the strain was smaller in beam 21-26-5 which had No. 5 bars as supplemental longitudinal reinforcement, than in the other beams which had No. 3 bars.

The posts below the loads and within the shear spans of the

beams which failed in strut flexure were reinforced by four vertical double-legged No. 3 stirrups; beams 25-16-6 and 26-21-7 also had four double-legged closed horizontal stirrups in the posts within the shear spans. Gages 5 and 6 on the vertical stirrups of posts 2 and 1, respectively, and gage 41 on the horizontal stirrups indicated in all cases the stirrups were adequate to resist the applied loads as the yield strain was not reached. The gages on the stirrups in the posts below the loads, gage 5 in beams 25-16-6 and 26-21-7 and gage 6 in beams 17-16-4, 20-26-5, 21-26-5 and 22-26-5 indicated small strains less than 0.001 in. Post 1 and gage 6 in beams 25-16-6 and 26-21-7 were completely within the shear spans and, therefore, the strains were much larger. The load strain curves for gage 6 in these beams were very similar to that of beam 19-16-6 which had similar post reinforcement but failed in flexure. For beam 25-21-6, the average force in the vertical stirrups for the last load increment was 1.83 kips per kip shear compared to 2.51 kips per kip shear for the other beams which failed in post shear with the same geometry. For beam 26-21-7, the axial force in the stirrup was 2.44 kips per kip shear compared to 2.42 kips per kip shear for beam 30-21-7 which failed in post shear. The average force in the horizontal stirrups calculated from the strain in gage 41 was 1.01 or 1.29 kips per kip shear for beams 25-16-6 and 26-21-7, respectively.

The solid shear spans of the beams failing in strut flexure were reinforced with vertical double legged No. 2 or 3 stirrups in different arrangements. Beams 17-16-4, 20-26-5, 21-26-5 and 22-26-5

had two No. 3 bars beside the hole and then No. 3 bars at 15 inches. Beams 25-16-6 and 26-21-7 had two No. 3 bars beside the hole and then No. 2 bars with spacings of 6 to 15 inches. The yield strain was reached at gage 7 in beams 20-26-5, 21-26-5 and 22-26-5 and at gage 8 in beam 17-16-4 but the cracks in no case opened greatly or reached the bottom of the beam.

The shear reinforcement in the struts above and below the holes in the shear spans of these beams was sufficient to cause strut flexural failures. The spacing of the closed vertical No. 2 stirrups varied from 1.50 to 2.75 inches in the top strut and from 1.50 to 2.75 inches in the bottom struts. The shear reinforcement of all of these beams was designed by section 11.4.3 of the ACI Code (1) with the shear proportioned to the top and bottom struts by shear area including the chamfers and part of the flange. An exception was beam 22-26-5 in which the shear was proportioned by the gross concrete area. This led to closer stirrup spacing in the top struts and longer spacing in the bottom struts of beam 22-26-5 than in beam 20-26-5 which was otherwise the same. Though there were more shear cracks and the recorded strains were larger with the larger spacings, both beams failed by the same strut flexural mechanism at the same load. The gages on the strut shear reinforcement which recorded the highest strains were at the centerline of the bottom strut and on the first stirrup near the load in the top strut, with one exception, beam 17-16-4 where the critical gage in the top strut was at the strut centerline. The yield strain was reached in the strut shear reinforcement of three of the

beams: in beam 21-26-5 in both top and bottom struts, in beam 22-26-5 in the bottom strut, and in beam 26-21-7 in the top strut.

The struts in the shear spans of the beams failing in strut flexure were reinforced with supplementary longitudinal reinforcement for both negative and positive bending. The behaviour of the reinforcement brings to light information concerning mechanism formation and high bond stresses.

Three different reinforcing arrangements were used in the struts of the beams of this group (beams failing in strut flexure). Beams 17-16-4, 20-26-5, 22-26-5 and 26-21-7 had ten No. 3 bars, 6 in the top strut and 4 in the bottom strut. In the top strut, 4 bars were placed at the top and 2 at the bottom, all running the full length of the beam. In the bottom strut, 2 bars were placed in the top and bottom of the struts. The supplemental longitudinal reinforcement in the bottom strut extended from the ends of the beams to the centerline of the first hole in the pure moment region except in beam 26-21-7 where they were cut off 1.0 feet beyond hole 2 towards the beam centerline and 1.3 feet beyond hole 1 towards the support. Beams 21-26-5 and 25-16-6 each had unique strut flexural reinforcing arrangements. Beam 21-26-5 had four No. 3 bars running the full length in the top of the top strut and two No. 5 bars were placed at the bottom. In the bottom strut, two No. 5 bars were placed at the top and bottom of the struts. All the No. 5 bars in both top and bottom struts extended from the end of the beam to the centerline of hole 2 in the pure moment region. Beam 25-16-6 had ten No. 2 bars as supplemental longitudinal

reinforcement in the same horizontal locations discussed above. The bars in the top strut ran the full length of the beam. In the bottom struts the bars were cut off 1.3 feet beyond hole 1 towards the support and beyond hole 2 towards the beam centerline.

Electrical resistance strain gages were mounted on the strut flexural reinforcement of all the beams of this group. Beams 17-16-4, 20-26-5, 21-26-5 and 22-26-5 had gages mounted in the tension and compression zones at both ends of the top and bottom struts. Beams 25-16-6 and 26-21-7 had gages mounted on the bottom reinforcement of the top strut at both ends of each of the two holes in the shear spans while gages were placed only in the strut flexural tension zones in the bottom struts.

The load strain plots for these gages indicated that the gages in the bottom of the top strut and in the strut moment tension zones in the bottom strut were critical (had the highest strains and yielded before failure). The strains were higher in the struts with larger axial loads. Gages in the other noncritical locations reached the yield strain only in beam 22-26-5.

The noncritical gages were located on the top bars in the top strut and in the strut flexural compression zones of the bottom struts. The strain in the reinforcement in the top of the top strut near the load increased slowly in compression to near the ultimate load but as the neutral axis moved upward above the bars, the gages indicated a small tensile strain. At the reaction end of top reinforcement in the

top strut, in the tension zone of the strut moments, the strain gages indicated a small compressive strain at early stages of loading but showed tensile strain at higher loads and reached the yield strain in beam 22-26-5. The reinforcement in the strut flexural compression zone of the bottom struts was at the top of the strut near the load and at the bottom near the reaction. The gages at these locations registered compressive strains during the early stages of loading. At higher loads, however, as the axial tension in the struts increased, the compression zone was reduced to an area of concrete beyond the reinforcement and the strain increased in tension. Beam 22-26-5 was an exception with the gage at this location indicating a compressive strain to failure, and reached the yield strain.

The critical gages all indicated that the yield strain had been reached before failure. The first gages on the strut flexural reinforcement to indicate yielding were in the bottom of the bottom strut nearest the load where both the beam moments and strut moments produce tension. For beams with two holes in the shear spans, this was gage 34. The load-strain plots for this gage were linear to a load of 8 kips with the slope at higher loads decreasing to failure. However, even beyond yielding the slope remained greater than zero. The strains were slightly smaller for beam 25-16-6 than for 26-21-7 which had larger holes. The strains were also smaller at the corresponding location in hole 1 (gage 28) where the beam moment is smaller. For the beams with one hole in the shear span, the first gage to indicate that the yield strain had been reached was gage 38. The load

strain curve for this gage in beam 17-16-4 was linear to a load of 8 kips; from 8 to 16 kips the strain rate increased gradually and was again linear between 16 and 25 kips reaching the yield strain at a load of 19 kips, above a load of 25 kips, the recorded strains were irregular. For beams 20-26-5, 21-26-5 and 22-26-5 the load strain curves for gage 38 were nonlinear to a load of 10 kips having a slowly increasing rate of strain. At loads above 10 kips, the strain increased in a linear manner to beyond the load at which the reinforcement yielded. The load strain curves for these three beams all fall below that of beam 17-16-4. The strains in beams 20-26-5 and 22-26-5 were almost exactly the same for any given loading and in both beams, gage 38 reached the yield strain at a load of 13 kips. The strain in beam 21-26-5 was less than in beams 20-26-5 and 22-26-5 because of the heavier strut flexure reinforcement. Gage 38 in this beam reached the yield strain at a load of 17 kips. The second gage on the strut flexural reinforcement of the beams of this group to reach the yield strain was also in the bottom strut, at the top of the strut near the reaction end of the hole with the highest axial loads. For the beams with one or two holes in the shear spans, the load strain curves for the gages at the bottom near the load are parallel to those of the gages at the top near the reaction. The strains are slightly smaller near the reactions because of the eccentricity of the prestressing strands in the bottom strut (0.25 inches above the centroid of the bottom strut). The third and fourth gages to indicate yielding in the supplemental longitudinal reinforcement of the holes with higher axial loads were at the bottom of the top strut in either the tension zone near the load or in the

compression zone near the reaction. The reinforcement in the compression zone was the third to yield in beams 17-16-4, 21-26-5 and 25-16-6 and the fourth to yield in beams 20-26-5, 22-26-5 and 26-21-7. The order of reaching yield in the strut flexure tension zone is the opposite to that described above. The behaviour of the reinforcement in the tension zone at the bottom of the top strut as shown by the load strain curves for gages 26 and 30, was similar for all beams of this group. The slope of the load-strain curves decreased to failure with a few irregularities after the reinforcement had reached the yield strain. The reinforcement at gage 30 reached the yield strain only in beams with 1 hole in each shear span. The strain at any given load at gage 30 for the four beams, with one hole in the shear spans, was least in beam 17-16-4 and greatest in beam 22-26-5. The load strain curve for 21-26-5 fell just below that of 17-16-4 and that of 20-26-5 was just above that of 22-26-5. In the compression zone at the bottom of the top strut near the reaction end of the struts with the higher axial loads, gage 28 or 32 reached the yield strain in compression before failure. The load strain curves for these gages had increasing rates of compressive strain to failure and maximum strains well beyond the yield strain reaching a maximum of ten times the yield strain in beam 17-16-4.

Strut flexural failures as shown in the cracking patterns of beams 20-26-5 and 22-26-5 were the result of the formations of four plastic hinges, one at each end of both top and bottom struts. The formation of these hinges was confirmed by the yielding of the reinforcement as indicated by the strain gages or visually by crushing in

the concrete. Yielding of the strut flexural reinforcement in the top strut gave a good indication of the load at which hinges form in the top strut. Yielding of the supplemental longitudinal reinforcement in the bottom strut was not a good indication because of the presence of the high strength prestressing strands which picked up additional load after the mild steel had yielded and prevented plastic rotation of the joints in the bottom strut. The first visible sign of hinge formation in all six beams which failed in strut flexure was spalling of the concrete from the bottom of the top strut near the reaction. This concrete crushing occurred at loads between 0.86 and 1.00 of the ultimate load. In the four beams in which all four hinges did not form before failure, the changing properties of the concrete cross-section in the top strut due to the crushing of the concrete at one or more of the hinge locations led to failure.

A reasonable prediction of the strut flexural capacity was obtained by calculating the load at which the first hinge was formed. This was done by assuming:

1. The shear is distributed to the top and bottom struts according to shear areas, including the chamfers and part of the flange.
2. The inflection points are located at the strut centerlines.
3. The axial force, in the top strut acts through its centroid, and in the bottom strut through the centroid of the tension reinforcement and is equal to the beam moment at the strut centerline divided by the distance between the forces and calculating:

   a) The axial load-moment interaction diagrams for each hinge location,

b) The axial load moment relationship for each strut,

c) The lowest load at which the two curves above cross.

Using this procedure the capacity of the beams failing in strut failure was conservatively predicted. The actual ultimate capacity was between 1.35 and 1.14 with an average of 1.27 of the calculated capacity.

As the strut flexural mechanisms form, the stresses at any level change from tension at one end of the strut to compression at the other, placing high demands on the bond between the longitudinal reinforcement and the concrete. The change in stress in the bottom reinforcement of the top strut was the largest for the strut configuration used in this series. At this location, the stress changed from yield in compression to yield in tension along the length of the strut and across a post when there was more than one post in the shear span. According to the provisions of the ACI 318-71 (1), the minimum post and hole length to develop yield in tension and compression for No. 3, 4 or 5 bars is 15.5, 18.0 and 21.9 inches, respectively. These limits were exceeded only in the posts of beam 26-21-7 of the beams failing in strut flexure. However, there was some longitudinal splitting in the concrete and some irregularities in the load strain curves for the longitudinal reinforcement that indicated some slip had occurred. Longitudinal splitting was evident along the bottom reinforcement of the top strut in beams 17-16-4, 21-26-5, 25-16-6 and 26-21-7. There were irregularities in the load strain curves for the longitudinal reinforcement of all the beams of this group. These irregularities were generally at loads above that at which yielding occurred and indicated a reduced rate of strain increase.

The strut flexural behaviour of the beams which did not fail in strut flexure indicated the strut flexural reinforcement provided was adequate to resist the applied loads although there were signs of strut flexural distress. This distress took the form of yielding of the strut flexural reinforcement, longitudinal splitting along the strut flexure reinforcement, and other signs of bond failure, and the formation of strut flexural mechanisms over more than one opening. The first 16 beams cast generally had eight No. 3 supplemental longitudinal reinforcing bars with 6 in the top strut and 2 in the bottom of the bottom strut. There were no gages placed on the strut flexural reinforcement of these beams. From the cracking and failure patterns of these beams, it can be seen that only in beams where the posts in the shear spans failed did strut flexural mechanisms form and in these cases the mechanism was over more than one opening. There was some longitudinal splitting along the bottom reinforcement of the top strut indicating the presence of high bond stresses. In the last 14 beams cast, the supplemental longitudinal reinforcement consisted of 10 bars in each shear span. Six of these were placed in the top strut and four in the bottom strut. The top layer of bars in the top strut generally consisted of four No. 3 bars; in the remaining three layers, two No. 2, 3, 4 or 5 bars were used. Electrical resistance strain gages were placed on the strut flexural reinforcement at critical locations. Six of these 14 beams failed in flexure, two failed in post shear and the remaining six failed in strut flexure. The strain gages mounted on the strut flexural reinforcement of the beams with two openings in the shear spans indicated a slight tendency towards the formation of a strut flexural mechanism over two openings in the top strut with the highest strains in the bottom of

the top strut in the tension zone near the load of hole 2 and in the compression zone at the bottom of the top strut at the reaction end of hole 1. In the bottom strut, the gages indicated that the formation of mechanisms proceeded independently in the struts below both holes. The first yielding occurred at the bottom of the bottom strut nearest the load. The gages in the other strut flexure tension zones of the bottom strut indicated yielding at successively higher loads towards the reaction. The four beams which failed in flexure with 4 foot shear spans each had one 8 by 16 inch hole in each shear span and identical strut flexural reinforcement; yet there were large differences in the recorded strains in the strut flexural reinforcement of the top strut. The most probable reason for the different behaviour was the extremely high bond stresses resulting from the use of No. 5 bars for strut flexural reinforcement and the short 16 inch holes. Though failure occurred at 1.17 times the calculated strut flexural capacity, only in beam 23-16-4 did the strain gages indicate strains large enough to form a hinge at the reaction end of the top strut. In the bottom struts, the load strain curves were very similar despite the differences in the strains in the top struts.

5.4 Cracking and Deflection

The cracking patterns and deflections of the beams of this series were independent of the failure mode; there were, however, differences due to loading, beam geometry and, to a small extent, the reinforcement. The order in which the cracks appeared at various locations in each of the beams was similar for all of the beams and in each of the tests, cracks appeared in the shear span before flexural cracks

appeared in the pure moment region. In some of the beams tested, these cracks opened up and led to failure. However, these cracks did not greatly influence the beam behaviour of the beam up to the service load based on flexural cracking. At about the flexural cracking load, the centerline deflections for the beams of this series were greater than the theoretical elastic deflections.

The cracking patterns and Table 4.1 indicated there was a general order in which cracks appeared at various locations under applied load. Prior to loading, there were cracks visible due to the prestress force and creep and shrinkage. The general order of appearance of loading cracks was:

1. Corner crack from the lower reaction end of the hole closest to the reaction,

2. Corner crack from the upper load end of the hole closest to the load,

3. Strut flexure crack bottom of the bottom strut nearest to the load,

4. Diagonal cracks in post 1,

5. Flexure crack in the pure moment region,

6. Strut flexure crack at the top of the top strut near the reaction.

As can be seen in the above cracking order, several cracks appeared in the shear spans prior to flexural cracking in the pure moment region. These cracks did not greatly influence the behaviour of the beams of this series up to the service load and would likely not appear in less shear sensitive beams. This was demonstrated by Ragan and Warwaruk's (9) test of a full sized T-beam with large web openings which

when loaded to the service load had no cracks. This beam with a 120 ft. simply supported clear span and a height of 4 feet was loaded with a uniformly distributed load. The holes were 8 feet long and the height of the holes varied from 23 to 8 1/2 inches.

At failure, the beams of this series had several cracks in the pure moment region and in each shear span. The solid shear spans had two or three diagonal cracks which started from the edge of the first hole and sloped downward towards the support but only in two cases (beams 24-16-6 and 28-16-4) did these cracks reach the bottom of the beam. The bottom struts in the shear spans had closely spaced shear and flexural cracks. At the ends of the struts, the cracks were vertical. The slope of the cracks increased towards the center of the strut where cracks were sloped at 45°. The top struts in the shear spans also had shear and flexural cracks but they were more concentrated at the load ends of the struts. Twenty-two of the beams of this series developed flexural cracks at the reaction end of the top strut. This type of crack appeared only above the hole nearest the reaction. In some beams inclined web shear cracks appeared at the strut centerline and in the beams which failed in strut shear it was the crack which opened. The top struts also had some longitudinal splitting along the strut flexural reinforcement due to the high bond stresses involved in anchoring this reinforcement. The posts in the shear span, both those completely within the shear span and those below the load, were cut by a diagonal crack which extended from one corner of the post to the other and horizontal flexural cracks. The opening of the diagonal cracks led to post shear failures in 9 of the beams of this series, however, the

horizontal flexural cracks did not lead to any failures. In the pure moment regions, flexural cracks extended from the bottom of the beam to within 1 inch from the top and were spaced from 2 to 4 inches apart. The height of the flexural cracks at failure depended on the failure moment. The crack width was generally largest at the cutoff of the supplementary longitudinal reinforcement and there was some longitudinal splitting in this region. This cracking at the cutoff of the supplementary longitudinal suggests that the cutoff should be staggered and the strut shear reinforcement should continue into the pure moment region.

Eight beams were tested with 4 foot shear spans. Seven of these had 8 by 16 inch rectangular holes spaced at 24 inches on center while the other beam had 8 by 16 inch parallelogram shaped openings. The order of cracking in these beams was slightly different in that post 1 cracked after flexural cracks appeared in the pure moment region because the load was applied directly above post 1. Little difference was noted in the cracking loads due to the number of prestressing strands. The hole geometry did affect the cracking and for the beam with parallelogram shaped openings the cracking loads at each location were higher than for the beams with rectangular holes. The first crack appeared at the lower corner of hole 1 near the reaction at an average shear of 7.0 kips corresponding to a moment of 336 in-kips. The first flexural crack occurred at an average shear of 14.9 kips when the centerline moment was 714 in-kips.

Three beams were tested with 5 foot shear spans, all of which

had 8 by 26 inch holes and 12 inch posts. The cracking loads associated with strut flexure were lower for these beams than for the beams with shorter holes. On the other hand, the post cracking load was high because of the large post size and the loading. The first cracks due to the applied load appeared at the lower reaction end of hole 1 when the shear reached an average 5.0 kips when the centerline moment was 300 in kips. The first crack in the pure moment region was visible at an average shear of 10.3 kips corresponding to a centerline moment of 620 in kips.

Fifteen beams were tested with 6 foot shear spans. All of these had two holes in each shear span and holes spaced at 24 inches, except for beam 24-16-6 which had only one hole in each shear span. Three different hole geometries were used in these fifteen beams: 9 had 8 by 16 rectangular holes, 4 had 8 by 12 inch rectangular holes and 2 beams had 8 by 16 inch parallelogram shaped holes. The order in which the cracks appeared was dependent on geometry. For the beams with 8 by 16 inch rectangular shaped openings, the order of crack appearance was the same as the general cracking order, while for beams with 8 by 12 inch holes, because of the large posts, the post cracking load was higher than the flex cracking load. For the beams with parallelogram shaped openings, post cracks occurred before any other cracks were visible. The first cracks appeared at an average shear of 6.7 kips corresponding to a centerline moment of 485 in kips. The flexural cracking load in the pure moment region was not affected by the hole geometry, however, increasing the number of prestressing strands did

increase the flexural cracking load slightly. The flexural cracking load for the four beams with four strands averaged 9.9 kips giving a centerline moment of 711 in kips while for the beams with five strands the average flexural cracking load was 10.6 kips corresponding to a centerline moment of 763 in kips. Strut flexure cracks at the top of the top strut near the reaction above hole 1 appeared in only 8 of the 15 beams with 6 foot shear spans. Seven of the 9 beams with 8 by 16 inch rectangular holes exhibited this type of cracking while neither of the two beams with 8 by 16 parallelogram shaped openings and only one of the 3 beams with 8 by 12 inch rectangular openings exhibited this type of cracking. The average strut flexure cracking load for the top of the top strut for the 7 beams with 8 by 16 inch rectangular holes was 15.7 kips and ranged from 11 to 22 kips while for the one beam with 8 by 12 inch openings the cracking load was 21 kips.

There were only 2 beams tested with 7 foot shear spans. They each had 2 holes, 21 inches long, in each shear span. The cracking order for these beams was slightly different than the general cracking order in that post 1 cracked before flexural cracks appeared at the bottom of the bottom strut in the shear spans. The first crack occurred at a load of 6 kips when the centerline moment was 505 in kips. Flexural cracking in the pure moment region occurred at an average shear of 8.5 kips while the centerline moment was 714 in kips.

A 7-point loading was applied to two beams of this series, each of these had 8 by 16 inch rectangular openings spaced at 24 inches. The order in which the cracks appeared was typical except that post 1

cracked after flexural cracking in the pure moment region due to the relatively low horizontal shear on this post and the application of the load above this post. The load at which the cracks appeared at the various locations did not appear to be greatly affected by the number of prestressing strands and the cracking load at some locations was higher for the beam with four strands than the beam which had five strands. The first cracks appeared at an average load of 1.75 kips per jack corresponding to a moment at the beam centerline of 483 in kips. The first flexural cracks at the beams' centerlines occurred at a load of 2.5 kips when the moment at this location was 690 in kips.

The maximum service load and the load at which the load-deflection curve becomes nonlinear for prestressed concrete beams is generally considered to be the load at which flexural cracking occurs. The theoretical flexural cracking moment can be calculated using the formula:

$$M_{cr} = \frac{I}{Y_b} (f_{sp} + f_{pe} - f_d)$$

Using this formula with the average splitting strength of the concrete for the beams of this series, and the properties of the gross concrete section, the flexural cracking moment can be calculated for different span lengths and primary flexural reinforcing arrangements. The theoretical flexural cracking moment for beams with four 7-wire prestressing strands and 20 foot spans was 625 in kips while the actual flexural cracking loads were between 8 percent low and 15 percent high. For beams with 5 prestressing strands and 20 foot spans the theoretical cracking moment was 705 in kips and the actual flexural cracking moment

was between 8 percent low and 22 percent high. For the beams with 5 strands and 16 foot clear spans the theoretical flexural cracking moment was 730 in kips while the actual cracking moment was between 18 percent low and 19 percent high.

The centerline deflections of the beams of this series near the flexural cracking load, at a moment of 720 in kips, were greater than the calculated deflections. The actual centerline deflections were 1.10 to 1.79 times the calculated deflections using the moment of inertia of the gross concrete section of a beam without holes and the elastic modulus obtained from tests on the concrete. The moment of inertia of the gross concrete section used in this series is 4583 in^4 which is 5 percent higher than the moment of inertia at a section through a hole. The modulus of elasticity obtained from tests on the concrete used in this series was 3.26×10^6 psi while the modulus of elasticity based on section 8.3.1 of the ACI Code (1) is 4.25×10^6 which is 30 percent higher than the test value.

The theoretical deflection, calculated using the gross moment of inertia and the test value of the modulus of elasticity for the concrete from this series, is within 2 percent of the actual deflection of J. Sauve's control beam (JS-1) in the linear portion of the moment-deflection curve. At a moment of 720 in kips the actual centerline deflection was 0.30 inches while the calculated deflection was 0.305 inches, a difference of less than 2 percent. The differences between the theoretical and actual deflections for E. LeBlanc's control beam was much larger than for Sauve's control beam. The actual deflec-

tion at a moment of 720 in kips was 0.46 inches, 1.64 times the calculated deflection of 0.28 inches. Though it does not explain why there is such a discrepancy, it can be seen from the moment-deflection curve for this beam that there was an irregularity in the slope of the curve at a moment of 220 in kips. For higher loads the deflection of this beam was greater than that of the two beams of this series with the same loading (7-point loading).

One of the two beams of this series with 7-point loading had lighter flexural reinforcement than EL-1 while the other had heavier flexural reinforcement. All three beams had the same clear span of 20 feet. EL-1 had four 7-wire 3/8 inch diameter prestressing strands and two No. 3 bars in the bottom of the beam. Of the two beams of this series, one had four strands while the other had five strands. For these two beams, the ratio of the actual to theoretical deflection was smaller for the beam with four strands, 1.10, while for the beam with five strands the ratio was 1.25.

Eight beams were tested with 4 foot shear spans; three of these had 20 foot clear spans and the remaining five had 16 foot clear spans. Of the beams with 20 foot spans, two had four prestressing strands, one of which had the shear capacity of the bottom strut reduced, while the other beam with a 20 foot span had five strands. The number of prestressing strands did not appear to affect the ratio of the actual to theoretical deflections which was 1.15 for both four and five strands. The deflection was much larger in the beam in which the shear capacity of the bottom strut was reduced and the actual

deflection was 1.45 times the theoretical deflection. The five beams with 16 foot clear spans all had five prestressing strands and the average ratios of the actual to the theoretical deflection was 1.35.

There were three beams tested with two point loads and 5 foot shear spans, each of these had 16 foot clear spans, 5 prestressing strands and holes 26 inches long; the longest used in this series. The average deflection at a moment of 720 in kips was 0.34 inches giving a ratio of actual to calculated deflection of 1.79, the largest for the beams of this series.

Fifteen beams were tested with 6 foot shear spans. Eleven had 20 foot clear spans and four had 16 foot clear spans. Four of the eleven beams with 20 foot clear spans had 4 prestressing strands and one of these had the shear capacity of the bottom strut reduced. The ratio of the actual to the theoretical deflection was 1.74 for the beam with the shear capacity of the bottom strut reduced while for the other three beams the ratio was 1.35. The remaining seven of the eleven beams with 20 foot clear spans had 5 strands and the ratio of the actual to theoretical deflection was 1.12. The four beams with 16 foot clear spans all had 5 prestressing strands and the actual deflection was 1.28 times the theoretical deflection.

There were two beams with 7 foot shear spans. These had 16 foot clear spans, 5 prestressing strands and holes 21 inches long. The average deflection of these beams was 1.63 times the theoretical deflection.

Several general observations were made concerning the actual deflection and the ratio of actual to theoretical deflection, at a moment of 720 in kips, in relation to the parameters of this series. The ratio of actual to theoretical deflection was larger for beams with longer holes being 1.79 and 1.63 for holes 26 and 21 inches, respectively, while for 12 or 16 inch long holes, there was little difference and the average ratio is only 1.23. Decreasing the clear span increased the ratio of the actual to theoretical deflections for beams with the same shear span hole size and flexural reinforcement. Reducing the shear span also increased the ratio of the actual to theoretical deflection. Reducing the shear capacity of the bottom strut as was done in beams 4-16-6 and 5-16-4 drastically increased the centerline deflections. Decreasing the number of holes in the shear span as was done in beam 24-16-6 slightly reduced the centerline deflection compared to other beams with the same loading clear span and flexural reinforcement. Changing the primary flexural reinforcement from 4 to 5 strands decreased the deflection of beams with 6 foot shear spans, did not affect the deflection of beams with 4 foot shear spans and increased the deflection of beams with 7-point loading. The reinforcement in the region around the holes had very little effect on the centerline deflection. There was only one case in which a change in the reinforcement around a hole measureably influenced the deflection at a moment of 720 in kips. Increasing the strut flexural reinforcement in the beams with 5 foot shear spans and holes 26 inches long decreased the deflection.

In the elastic range, the deflection of beams with multiple

holes were larger than the theoretical deflections of a beam without holes. Several cracks were evident in the beams of this series prior to flexural cracking in the pure moment region. However, these did not affect the behaviour nor would occur before flexural cracking in less shear sensitive beams.

CHAPTER 6

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary

Thirty simply supported prestressed concrete T-beams containing large web openings were tested to examine the behaviour and develop design procedures for such beams. The behaviour of these beams is recorded in terms of applied loads and the resulting deformations. The prime variable in this study is the reinforcement required in the region around the holes. Other parameters such as geometry, loading conditions, and flexural capacity were varied to place different demands on the reinforcement in the region of the holes.

The reinforcement in the region of a hole is grouped into four types; post reinforcement, solid shear span shear reinforcement, strut shear reinforcement, and strut flexural reinforcement. Several different arrangements were used for each type. The post reinforcement was provided by vertical or inclined double-legged stirrups. In some beams, supplementary post reinforcement consisting of horizontal stirrups or inclined bars was added to the vertical stirrups. In the solid shear spans, the shear reinforcement consisted of vertical or inclined double-legged No. 2 or 3 stirrups at various spacings. The strut shear reinforcement was closed and No. 2 stirrups set at various spacings. The strut flexural reinforcement was provided by supplementary longitudinal reinforcement at four levels in the cross-section, at the top and bottom of the top and bottom struts, and consisted of

straight No. 2, 3, 4, or 5 bars. Several aspects of the beam geometry were varied in these tests. The horizontal dimension of the holes varied from 12 to 26 inches while that of the posts varied from 8 to 12 inches. Twenty-seven of the beams had rectangular holes and vertical posts. The other three beams had parallelogram shaped holes and inclined posts. The clear span length was reduced from 20 to 16 feet to facilitate casting. Five different symmetrical loadings were used. One was a 7-point load while the remaining were 2-point loads with 4, 5, 6, or 7 foot shear spans. The flexural capacity was varied by using four or five 7-wire, 3/8 inch, stress relieved, 250 K prestressing strands as primary flexural reinforcement. These tests resulted in 10 flexural failures, 9 post shear failures, 5 strut shear failures, and 6 strut flexural failures.

6.2 Conclusions

The following conclusions are based on the results of this investigation.

1. Beams with large web openings can be designed to resist large shear forces and to fail in flexure.
2. Beams with large holes require extra reinforcement in the region around the holes particularly in regions of shear.
3. Large holes in the pure moment region do not affect the strength but reduce the stiffness of a beam. Extremely long holes should be checked for stability.
4. Adequately reinforced beams with large web openings behaved as typical under reinforced prestressed concrete T-beams with a linear

moment deflection curve to the flexural cracking load and large deflections and closely spaced flexural cracks at the ultimate load.

5. Beams with large holes are not as stiff as beams without holes.
6. The flexural capacity was conservatively predicted with an error of less than 16 percent using the approximate formula in the ACI Code (1) for the stress in the prestressing strand at design load. Strain compatibility analysis was more accurate in that rupture of the prestressing strand could be predicted.
7. Beams with large transverse holes in the webs exhibit Vierendeel truss behaviour.
8. The shear on a section through a hole is distributed in some changing ratio to the top and bottom struts. When the applied moment is small, the bottom strut carries a large portion of the shear. As the moment increases, the proportion of shear carried by the bottom strut reduces. In this series, satisfactory design was accomplished assuming the shear was proportioned by shear areas which included the contribution of the flange and chamfer to the shear area of the top strut.
9. The struts, where shear is present, are subject to bending moments which result from the strut shear acting at a point of inflection. Assuming this inflection point is at the strut centerline is an accurate assumption for the bottom strut which is essentially symmetrical. However, for the top strut which is neither symmetrical nor symmetrically reinforced, this assumption is conservative. This is because the flexural capacity at each end of the strut is different and the point of inflection moves away from the end with the higher capacity so that both ends fail near the same load (which is

higher than that predicted using the smaller capacity with the inflection point at the strut centerline).

10. A definition of a large web opening is not practical and the effect of all web openings (except small pipe sleeves) should be checked for each application particularly where shear is present or possible.
11. Posts in beams with multiple openings where shear is present are subjected to a horizontal shear resulting from change in moments between the centerlines of the holes to either side, moments due to the horizontal shear and changes of the moments in the struts, and an axial load due to directly applied loads and changes in strut shear.
12. The horizontal shear produced extremely large shear stresses on the horizontal cross-section of the posts; as high as 1245 psi on some of the posts of this series.
13. Post reinforcement had the greatest influence on the post shear capacity:
    a) Increasing the total area of vertical double-legged stirrups increased the post's ability to resist the applied forces. The axial load on vertical post stirrups was less than 1.56 times the horizontal shear.
    b) Inclining the post reinforcement at 45° increased the post capacity compared to the same area of vertical stirrups. The axial force on the inclined stirrups was less than 1.10 times the horizontal shear.
    c) Adding horizontal stirrups to vertical post stirrups increased the post's capacity and reduced the strain in the vertical

stirrups. The axial loads on the vertical and horizontal stirrups were less than 1.24 and 0.99 times the horizontal shear.

14. Increasing the horizontal dimension of a post increased the post's capacity and the post cracking load.
15. The horizontal shear increases with hole spacing; however, from these tests the strain in the post reinforcement was higher only near the ultimate load.
16. Increasing the axial load on a post by application of load directly above a post increased the capacity of the post.
17. Decreasing the axial load on a post by reducing the shear capacity of the bottom strut reduced the capacity of the post.
18. The moments on the posts of the beams of this series were adequately resisted by distribution of the shear reinforcement over the horizontal dimension of the post.
19. The struts above and below a hole where shear is present are subjected to shear flexure and axial loads.
20. The distribution of the shear to the top and bottom struts is critical. No strut shear failures occurred in beams in which the shear was proportioned by shear area including the contribution of the flange and chamfers and reinforced accordingly.
21. The shear is critical in the beams of the proportion used in this series through a hole where the beam moment is largest.
22. The concrete in struts without strut shear reinforcement can resist considerable shear.
23. Strut shear reinforcement has a major effect on the strut shear capa-

city. Adding strut stirrups to the struts increased the ability of the struts to resist shear.

24. Decreasing the spacing of the strut stirrups increased the strut's ability to resist shear.
25. Inclining the strut stirrups at 45° slightly reduced the strain in the strut stirrups.
26. The critical shear crack in the top strut was a web shear crack at about the centerline of the opening.
27. Inclination of the post stirrups increased the top strut shear capacity.
28. As the hole length was increased, the strut shear capacity was reduced.
29. Increasing the flexural capacity of the beam by increasing the number of prestressing strands slightly increased the shear capacity of the bottom struts by reducing the axial tension acting on the bottom struts.
30. The supplementary longitudinal reinforcement had a great influence on the strut flexural capacity.
31. The major factors affecting the strut moments are distribution of shear to the struts, location of the inflection points, and the hole length.
32. Increasing the area of supplementary longitudinal reinforcement increased the strut flexural capacity.
33. Anchorage of the strut flexural reinforcement is a problem in beams subjected to large shears which have holes and posts with small horizontal dimensions.

34. The solid shear spans of beams with large web openings can be designed in the usual manner as long as extra stirrups (to carry the entire shear on the section) are placed beside the first hole.
35. All the beams of this series had shear cracks in the shear spans before flexural cracking in the pure moment region. However, for less shear sensitive beams, no such problem would exist.
36. The cracks which occurred before flexural cracking did not influence the beam behaviour up to the service load.
37. The deflections of the beams of this series at a moment of 720 in-kips were between 1.10 and 1.79 times the theoretical deflections calculated using the moment of inertia of the gross concrete section and the test value of the modulus of elasticity for the concrete.
38. The ratio of the actual to theoretical deflection is larger for beams with: (i) longer holes, (ii) shorter clear spans, (iii) shorter shear spans, and (iv) the shear capacity of the bottom strut reduced.
39. The ratio of the actual to theoretical deflection is decreased by reducing the number of holes (as was done in beam 24-16-6).

6.3 Recommendations

6.3.1 Design Procedure

1. Design for flexure using ACI Code (1) approximate formulas or strain compatibility.
2. Design solid shear span in usual manner using ACI Code (1) formulas

and placing extra stirrups beside the first hole to carry the total shear on the section.

3. Assume the beam in the region of a hole behaves as a Vierendeel truss.
4. Assume the shear is proportioned to the top and bottom struts in proportion to their respective shear areas.
5. Assume points of inflection in the struts are located at the centerline of the strut length.
6. The axial load on the struts due to the applied loads is equal to the bending moment at the centerline of a hole divided by the distance between the primary tension reinforcement and the centroid of the top strut.
7. Design the struts for these forces using the ACI Code procedures for shear and axial load, and the strut interaction diagram to check the strut flexural and axial load capacity.
8. Design the posts to resist the horizontal shear due to the change in the strut axial loads on either side of the post. This can be accomplished using the required area of reinforcement as suggested from the tests of this series for the various arrangements and distributing it evenly across the post.

6.3.2 Further Investigation

1. Investigate the behaviour of struts subjected to transverse loading.
2. Investigate the behaviour of struts long enough to buckle.
3. Investigate and develop a relationship for the distribution of shear to the top and bottom struts which should include (i) shear area,

(ii) applied shear and (iii) the ratio of the applied to ultimate moment.

4. Further investigation in post behaviour in relation to deep beam action.
5. Investigate the use of the prestressing strands to reinforce the bottom strut against strut moments where the strand is not required to resist beam moments, i.e., near the supports in a simply supported beam. This is normally a poor location for a hole, however, it might lead to more efficient use of the prestressing strand.
6. Investigate the effective hole length to be used in calculating strut moments for hole shapes other than rectangular.

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APPENDIX - A

A.1 Materials

1. Cement

Type III, high-early strength, portland cement was used in all mixes.

2. Aggregate

Two aggregates were used in each mix; a fine sand and a coarse aggregate. The sand was a well graded fine sand with a fineness modulus of 2.53. The coarse aggregate was 3/8 inch pea gravel. Both of these aggregates were used in several previous investigations and always gave satisfactory results.

3. Concrete Mix

A satisfactory mix had been developed in the laboratory using these materials. This mix was used in this series. The proportions used were as follows:

Cement	1.0
Sand	2.2
Coarse aggregate	1.6
Water	0.39 - 0.51

The proportion of water added was varied to give a constant water/cement ratio accounting for slight variations in the moisture content of the aggregate. The slump was maintained between 3 and 4 inches to facilitate compaction of the concrete around the congested reinforcement in the posts and below the voids.

Three batches were required for each beam with a 20 foot clear span while only two were required for each beam with a 16 foot clear

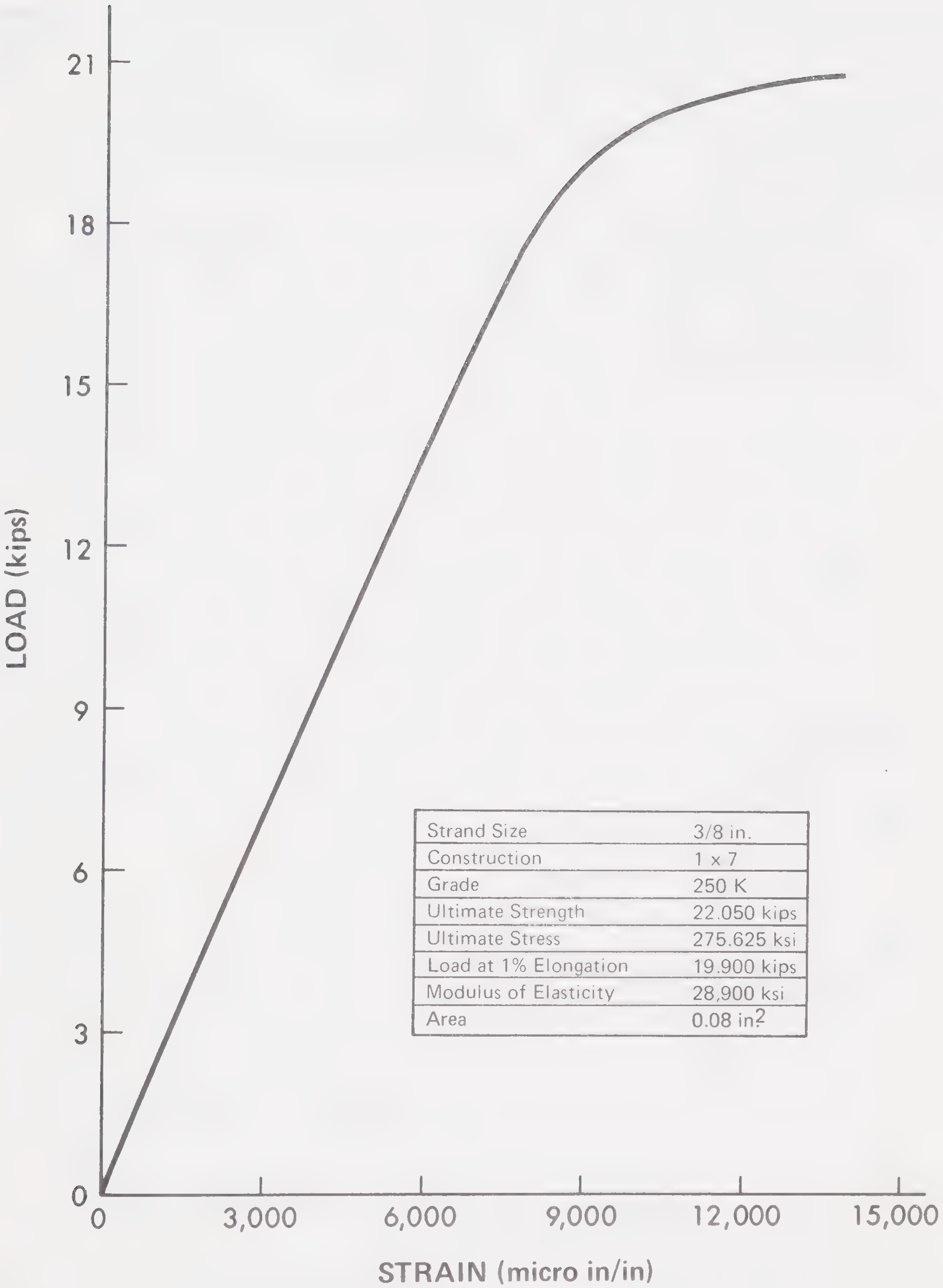
span. For each batch, three 6 by 12 inch cylinders were cast and subjected to the same curing as the beams and were tested at the time of the beam tests. Two cylinders from each batch were tested in compression and the third was used to determine the tensile splitting strength. The average compression strength was 5554 psi and varied from 6348 to 4991. The average splitting strength was 410 psi and varied from 301 to 548 psi. The modulus of elasticity was calculated from the load and deformation over an 8 inch gage length for ten of the cylinders used in the compressive strength tests for five beams. The average elastic modulus was 3.26×10^6 psi. ASTM specifications were followed in the sampling, molding and testing of the concrete. Table A.1 presents the age at testing the average compressive strength and the average splitting tensile strength of the concrete used in each beam.

4. Prestressing Strand

The prestressing strand used in the test beams was 250 K grade, 7-wire, stress relieved strand, with a nominal diameter of 3/8 inch and complied with ASTM A-416 specifications. The load-strain curve for this strand was obtained from the manufacturer's test and is duplicated in Figure A.1. The shape of the load-strain curve was confirmed by a test up to 80 percent of the ultimate tensile strength. The strain gages used in this test were mounted as they were in the test beams, that is, on one of the six curved wires of the 7-wire strand, and oriented along the axis of that wire at approximately 8°30' from the long axis of the strand. Above a load of 80 percent of the ultimate tensile strength the teeth on the conical wedge grips

TABLE A.1 SUMMARY OF CONCRETE STRENGTHS

Beam No.	Avg. Cylinder Strength (psi)	Avg. Splitting Strength (psi)	Age at Test (Days)	Modulus of Elasticity (x 10^6 psi)
1-16-6	5821	476	28	3.17
2-16-4	5906	424	28	3.50
3-16-6	5296	424	21	3.17
4-16-6	5085	371	20	3.06
5-16-4	5409	392	15	3.41
6-16-6	5529	301	22	
7-16-6	6348	368	23	
8-16-7L	5255	367	23	
9-16-7L	5709	365	21	
10-16-6	5741	392	18	
11-16-4-P	6192	447	23	
12-16-6-P	5833	404	23	
13-16-6-P	5394	401	22	
14-12-6	5706	410	23	
15-12-6	5853	435	23	
16-12-6	5883	436	23	
17-16-4	5542	466	20	
18-16-4	5933	385	22	
19-16-6	5875	367	21	
20-26-5	5690	528	27	
21-26-5	5517	438	20	
22-26-5	5385	360	19	
23-16-4	4991	422.2	19	
24-16-6	5226	497.4	21	
25-16-6	5517	375.8	15	
26-21-7	5274	340.4	19	
27-16-4	5378	548.2	40	
28-16-4	5469	481.9	41	
29-12-6	5080	331.6	20	
30-21-7	4766	340.4	22	
AVERAGE	5554	410		3.26



Strand Size	3/8 in.
Construction	1 x 7
Grade	250 K
Ultimate Strength	22.050 kips
Ultimate Stress	275.625 ksi
Load at 1% Elongation	19.900 kips
Modulus of Elasticity	28,900 ksi
Area	0.08 in^2

FIGURE A.1. Load-Strain Curve and Properties of Prestressing Strand

tended to bite into the wires sufficiently to cause failure of one wire.

5. Shear Reinforcement

The main web reinforcement used in the solid shear spans and posts were fabricated from No. 2 or 3 bars. These stirrups made of No. 2 bars were bent to the required shape in the laboratory while the No. 3 stirrups were bent by the steel supplier. The strut stirrups were formed from No. 2 bars in the laboratory. The horizontal stirrups in the posts were fabricated in the laboratory from straight No. 2 or No. 3 bars and welded closed to reduce the number of bends required and slightly relieve the congestion in the posts. The idealized stress strain curves and properties given in Figure A.2 are the result of tests conducted in the Baldwin testing machine on three specimens of each group. The No. 3 main stirrups had the properties of the No. 3 bar in Figure A.2 while the No. 3 bar used in the horizontal post stirrups had the same properties as the No. 3 supplementary longitudinal reinforcement as shown in Figure A.3. The No. 2 bar used in beams 1-16-6 to 22-26-5 inclusive is designated in Figure A.2 as "No. 2a" while the No. 2 bar used in beams 23-16-4 to 30-21-7 is designated in Figure A.2 as "No. 2b".

6. Supplementary Longitudinal Reinforcement

The supplementary longitudinal reinforcement consisted of No. 2, 3, 4 and 5 bars. The idealized load-strain curves and the properties of the No. 3, 4 and 5 bars are found in Figure A.3. The

Bar Size		No. 2a	No. 2b	No. 3
Load at Yield	kips	2.216	1.690	6.077
Yield Stress	ksi	44.3	33.8	55.2
Yield Strain	micro in/in	1470	1150	1920
Elastic Modulus	ksi	30,100	29,400	28,900
Ultimate Strength	kips	2.887	2.743	9.000
Area	in^2	0.05	0.05	0.11

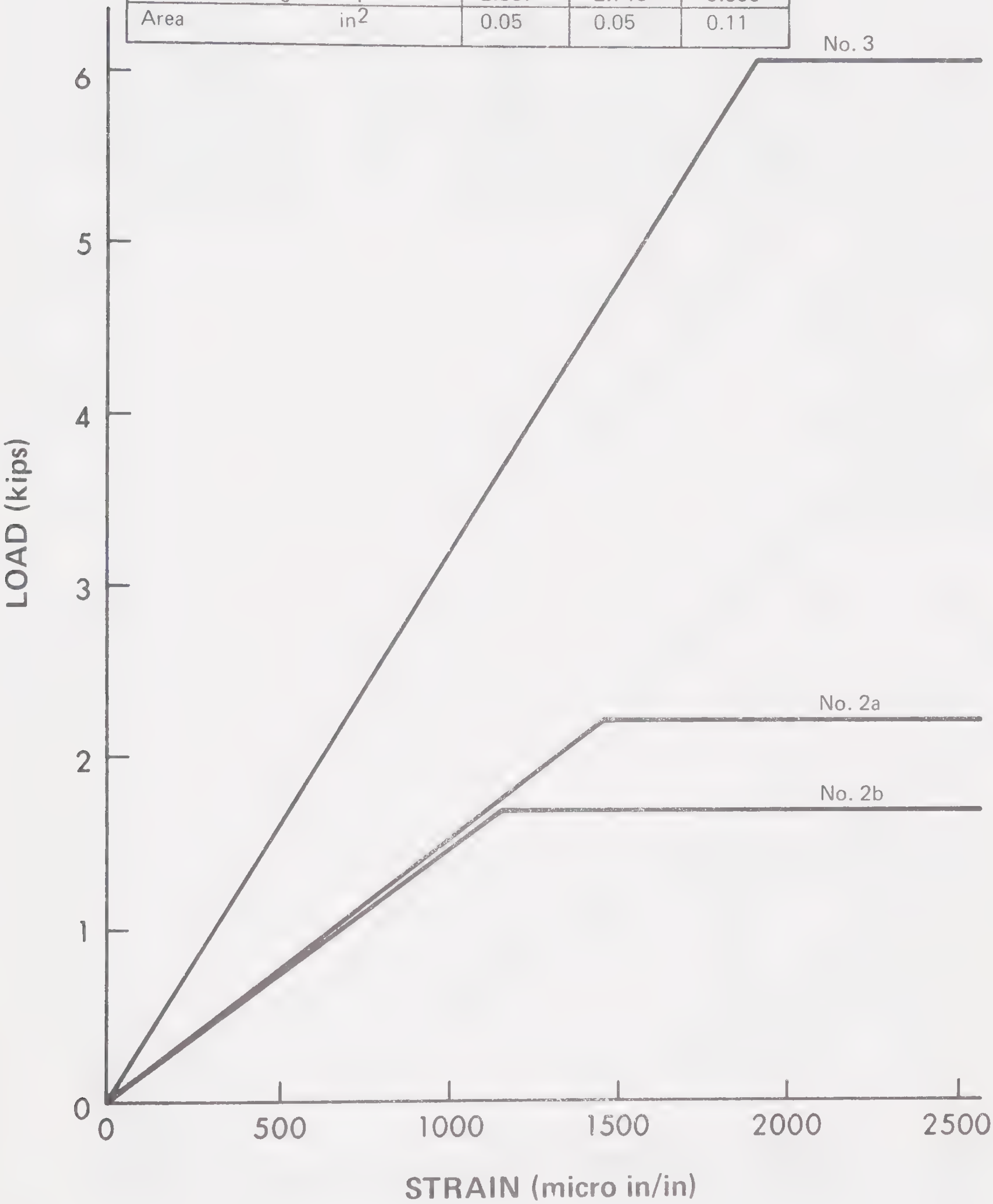
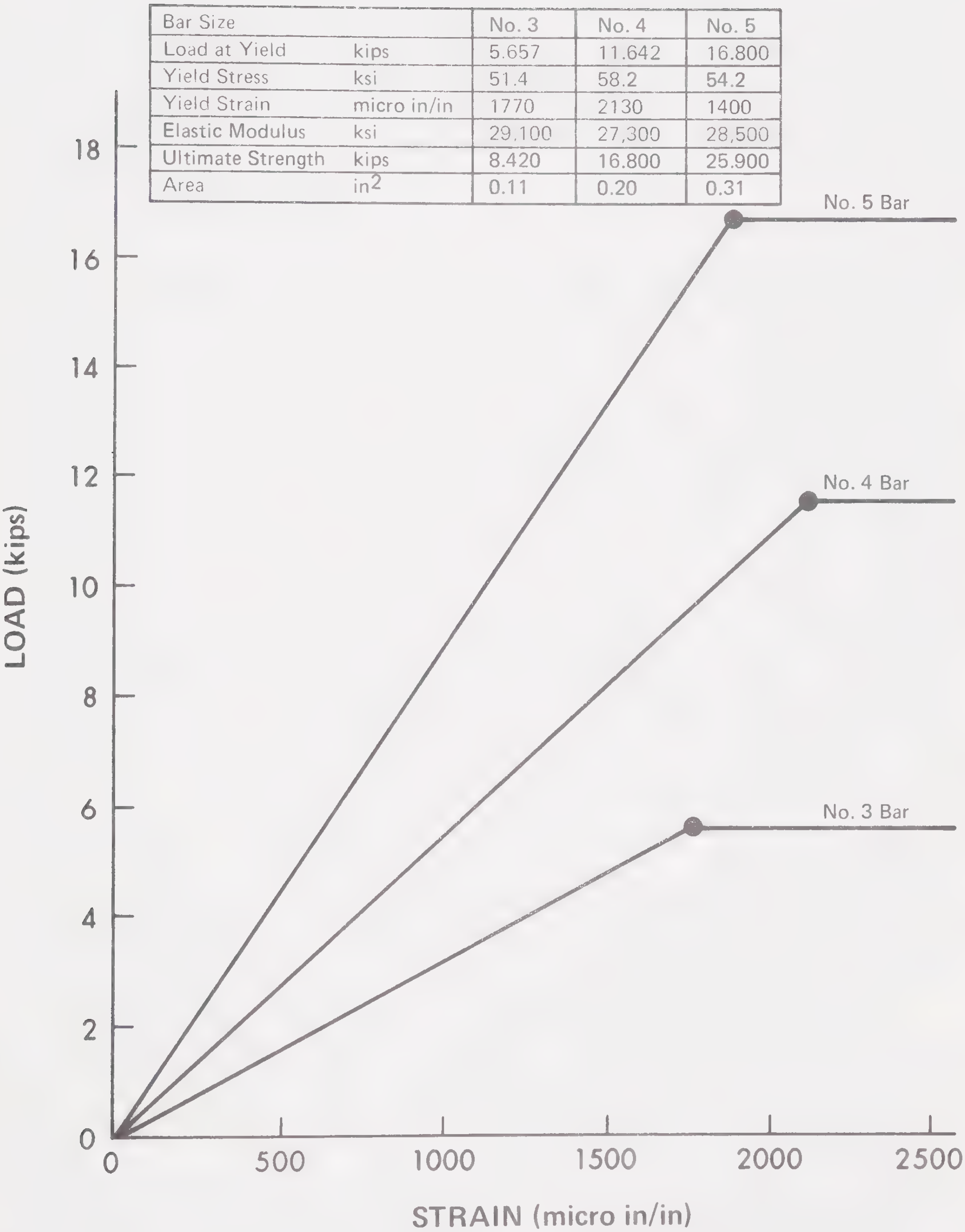


FIGURE A.2. Load-Strain Curves and Properties of Shear Reinforcement



Bar Size		No. 3	No. 4	No. 5
Load at Yield	kips	5.657	11.642	16.800
Yield Stress	ksi	51.4	58.2	54.2
Yield Strain	micro in/in	1770	2130	1400
Elastic Modulus	ksi	29,100	27,300	28,500
Ultimate Strength	kips	8.420	16.800	25.900
Area	in^2	0.11	0.20	0.31

FIGURE A.3. Load-Strain Curves and Properties of Supplementary Longitudinal Reinforcement

No. 2 bars used were from the same lots as the No. 2 bar shear reinforcement and the load-strain curves and properties are found in Figure A.2.

A.2 Fabrication

1. Form Work

Thirty feet of steel forms with 1/2 inch plywood liners were used to cast 1 beam with a 20 foot clear span and two beams with 16 foot clear spans were cast in tandem using 40 feet of forms. The forms were fabricated in ten sections of 1/8 inch mild steel plate, stiffened along the edges, with 2 1/2 by 1 1/2 by 3/16 inch angles and transversely by 1 3/4 by 1 1/4 by 1/8 inch angles at 2 feet on center. Each half of the section was bolted at 2 foot centers to a 4 by 1 5/8 by 3/16 inch channel which formed the bottom surface of the stem. The web openings were formed with styrofoam blocks cut to the desired shape and held in place within the form by conical void anchors. The conical void anchors were 1 1/2 inch high and had a diameter which tapered from 3 3/16 to 2 7/8 inches and were attached to the plywood liners with wood screws. The conical void anchors mated with 3 inch holes drilled in the styrofoam. This method of forming and anchoring the voids worked well and made changing the size, shape and spacing of the voids easy. The 1/2 inch plywood liners were attached to the steel forms with 1/8 inch bolts. The bolt heads were set into the wood and puttied over with auto-body putty. The liner was then painted with a plastic surface coating which gave the liner great durability and resulted in

an excellent finish on the concrete. The forms sat on 4 by 4 by 1/4 inch hollow structural steel tubes and were held in lateral position by turnbuckles attached to the top and bottom of the forms at 10 foot intervals. The end view of a form section is shown in Figure A.4 and photos of the forms are presented in Figure A.5.

2. Prestressing

The tensioning of the prestressing strands was completed on the stressing bed which consisted of two steel and reinforced concrete abutments which were anchored to the test bed and resisted the tension in the prestressed strands. The prefabricated cages of supplementary longitudinal and shear reinforcement, tack welded together with the strain gages in place and waterproofed, were set into the form with one side removed for accessibility. The prestressing strands were then threaded through anchor plates, the abutments and the cages. At one end, dynamometers were slipped on to each cable before the conical wedge grips used to anchor the cables were set into place.

Each strand was tensioned individually using a simplex center-hole jack operated by an electric Blackhawk pump. The load was measured by the dynamometer and checked using the pump line pressure. When the correct tension in each strand was reached, the conical wedge grip located between the jack and the anchor plate was pushed snug against the anchor plate and the jack was released. After the prestressing was completed, gages were mounted on the prestressing strands and waterproofed, the voids were set in place, the form sections were set in place and positioned bolted and aligned, and were now ready for casting.

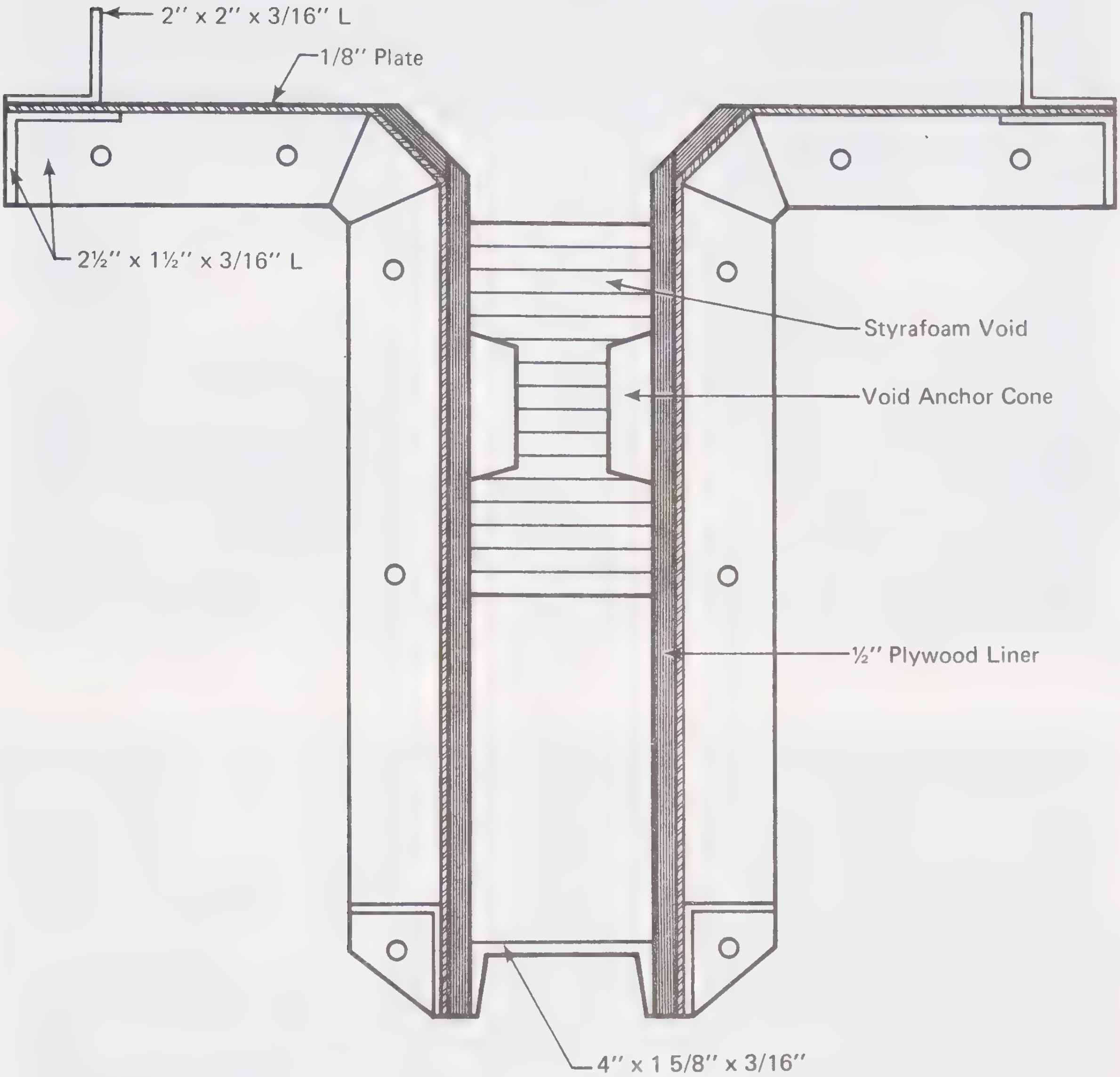


FIGURE A.4. Typical Form Cross Section

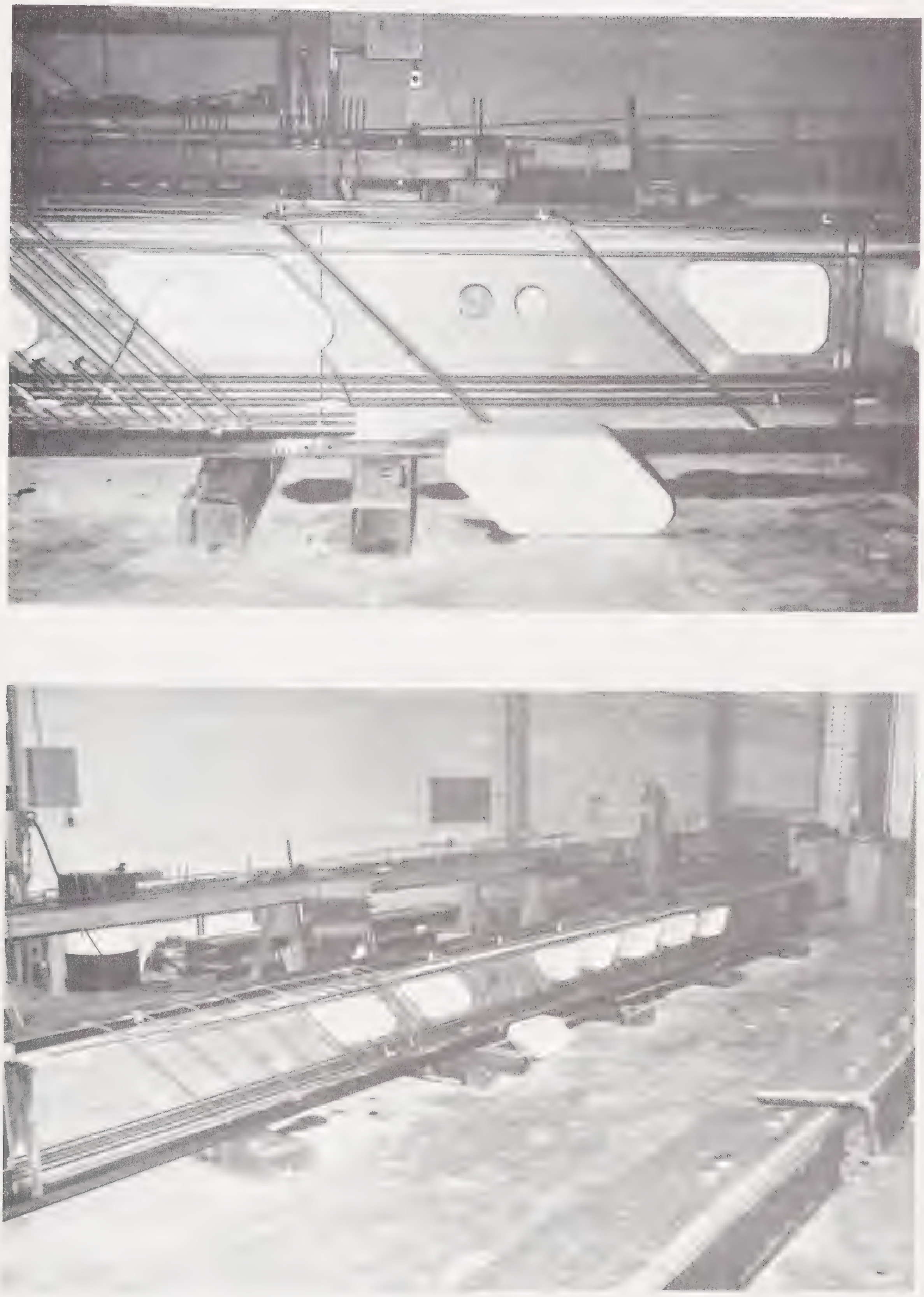


FIGURE A.5 FORMS WITH ONE SIZE REMOVED

3. Casting and Curing

The concrete was mixed in the laboratory's vertical drum, 8 cu.ft., mixer. The concrete was shovelled into the form and compacted using an immersion type vibrator. Three 6 by 12 inch test cylinders were cast from each batch. Twenty-four hours after casting, the side forms were stripped and the beam and the test cylinders were covered with wet burlap and a polyethylene tarp to prevent evaporation. The moist curing was continued until the prestressing strands were cut. After release of the strands the beam and test cylinders were stored in the laboratory atmosphere until they were tested.

4. Release of Prestress

The prestressing force was released to the beams five days after casting. The prestressing force was released by slowly applying heat using an oxyacetylene torch to the strands between the abutments and the beam. The fracture was gentle, indicating that a uniform transfer of prestress had resulted.

A.3 Prestress Losses

A complete set of Demec strain gage readings were taken at the beam centerline immediately before and after release of the prestress and at the beginning of each test. These readings were used to calculate the loss of strain in the prestressing strand. From these strains, the initial prestress force (measured just prior to release) and the modulus of elasticity of the prestressing strand, the prestress

losses were calculated. The initial prestress force, the effective prestress force and the total losses are listed in Table A.2. These "total losses" include elastic, creep and shrinkage losses of the concrete and approximately 20% of relaxation losses of the prestressing strands. However, the "total losses" do not include losses which occurred during the prestressing operation and between prestressing and release of the prestress, that is, losses due to slip of the anchors, release of the jack and approximately 80% of the relaxation of the prestressing strand.

A.4 Loading Apparatus

The symmetrical two and seven-point loadings used in this series were applied with loading harnesses. Each harness consisted of two 4 by 4 by 1/4 inch hollow structural steel tubes and two 3/4 inch high strength steel rods 8 feet long.

One of the tubes rested transversely across the flange of the beam and the other was suspended below the test floor on the steel rods. The jacks were mounted on the lower tube, the test floor provided the reaction to the jacking force. A typical cross-section of this loading setup is shown in Figure A.6. Seven 10 ton hydraulic jacks were used to apply the seven-point loads. The two-point loads were applied by two 30 or 50 ton hydraulic jacks. For beams with 5, 6 or 7 foot shear spans, two 30 ton jacks were used. For beams with 4 foot shear spans up to and including beam 11-16-4-P, the 30 ton jacks were also used. However, for the remaining beams with 4 foot shear

TABLE A.2 SUMMARY OF PRESTRESS LOSSES

Beam No.	No. of Strands	Initial Prestress (kips)	Total Losses (% of P_I)	Effective Prestress (kips)
1-16-6	4	57.14	17.4	47.26
2-16-4	4	56.89	17.4	47.01
3-16-6	5	68.79	26.2	50.78
4-16-6	4	56.85	21.0	44.93
5-16-4	4	56.94	19.6	47.77
6-16-6	4	57.30	21.6	44.91
7-16-6	5	69.81	26.1	51.62
8-16-7L	4	56.41	19.1	45.63
9-16-7L	5	70.98	24.0	53.91
10-16-6	4	57.83	19.3	46.70
11-16-4-P	5	72.05	25.2	53.90
12-16-6-P	5	71.42	24.6	53.85
13-16-6-P	5	71.51	27.0	52.20
14-12-6	5	71.50	24.8	53.77
15-12-6	5	71.86	21.6	56.37
16-12-6	5	71.58	23.8	54.52
17-16-4	5	70.97	23.5	54.33
18-16-4	5	71.17	25.2	53.25
19-16-6	5	70.25	26.2	51.87
20-26-5	5	70.25	27.5	50.94
21-26-5	5	70.31	27.9	50.67
22-26-5	5	70.79	28.6	50.57
23-16-4	5	70.92	24.3	53.70
24-16-6	5	70.92	20.1	56.65
25-16-6	5	71.23	20.6	56.55
26-21-7	5	71.23	23.0	54.82
27-16-4	5	72.00	33.6	47.84
28-16-4	5	72.00	29.7	50.62
29-12-6	5	71.18	24.4	53.84
30-21-7	5	71.18	27.4	51.72

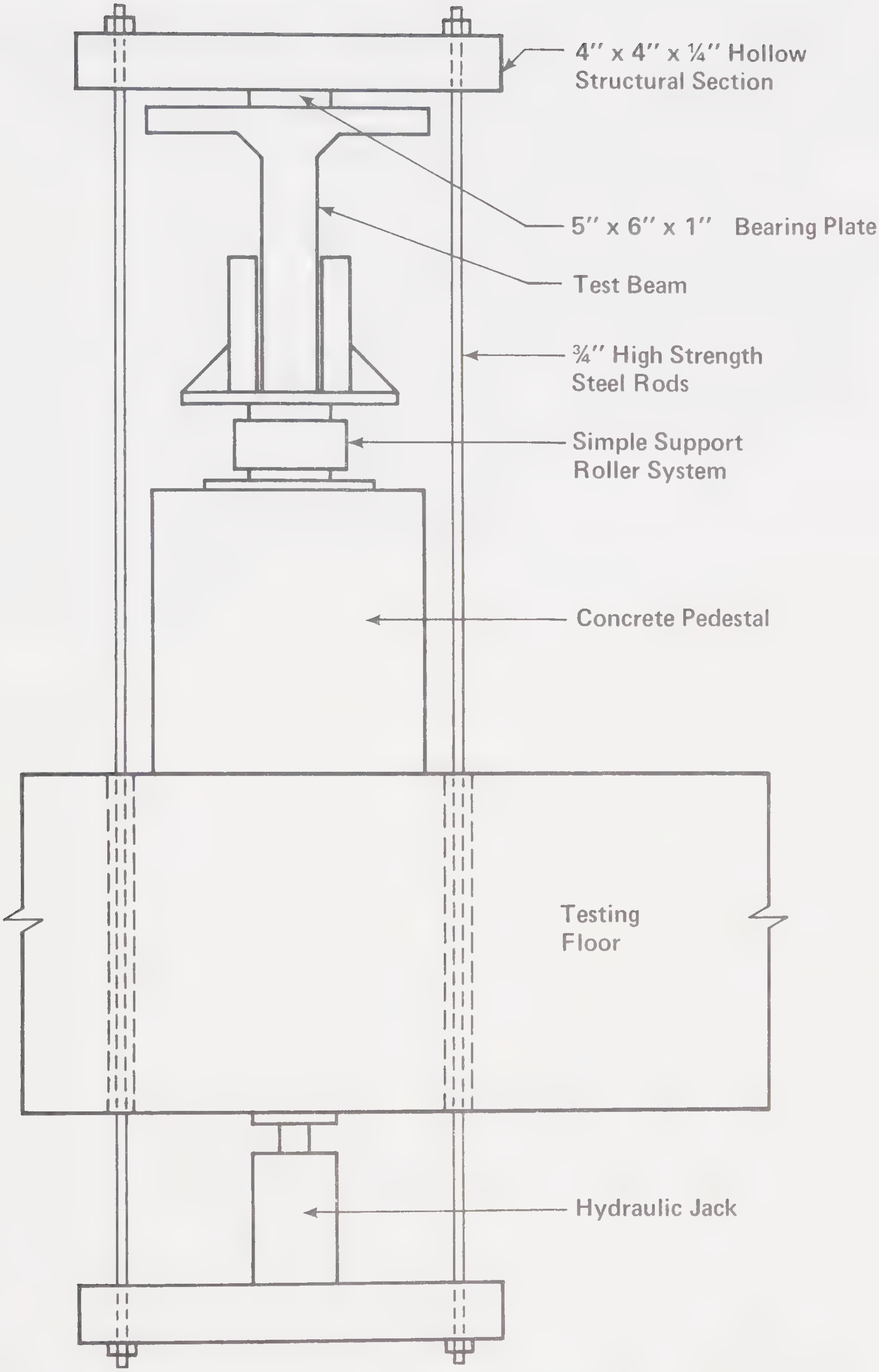


FIGURE A.5. Typical Cross Section of the Loading Setup

spans, two 50 ton rams were used. The hydraulic pressure to the jacks was provided by an Amsler pendulum dynamometer which pumped oil through a manifold then through separate lines to each jack.

Point loading on the longitudinal centerline of the beam was achieved using 5 by 6 by 1 inch steel bearing plates below each of the hollow structural steel tubes. Plaster of Paris was used to hold the bearing plates in position and provide an even bearing surface. Both beam supports were hinged to permit rotation. One of the supports was longitudinally fixed and the other was mounted on rollers to permit simple beam action.

APPENDIX - B

TEST DATA

TABLE B.1.1 ELECTRICAL STRAIN GAGE MEASUREMENTS (MICRO INCHES PER INCH)

BEAM NO. 1-16-6

Inc. No.	Load per Jack	Strain Gage Number								
		1	2	3	4	5	6	7	9	10
1	.148	0	0	0	0	0	0	0	0	0
2	1	25	20	20	10	- 5	0	0	0	0
3	2	60	50	40	25	-10	0	0	5	0
4	3	95	85	65	40	-10	10	0	5	0
5	4	130	120	90	50	-15	25	0	10	- 5
6	5	170	155	120	65	-15	35	0	5	- 5
7	6	210	190	150	80	-15	35	0	0	- 5
8	7	250	225	180	100	-15	45	25	0	- 5
9	7.5	270	245	195	105	-15	50	45	- 5	-10
10	8.0	290	260	210	110	-15	55	115	- 5	-10
11	8.5	325	285	235	120	-15	75	200	-10	-10
12	9.0	360	310	265	135	-10	115	285	-10	15
13	9.5	385	330	310	150	- 5	1140	320	-15	-15
14	10	410	350	325	160	0	1225	330	-15	-20
16	11	800	430	585	190	30	1400	395	-15	-20
18	12	1300	1720	830	210	45	1525	460	0	-25
19	13	1860	2380	1130	245	50	1640	525	25	-30
20	14	3310	3180	1385	410	55	1785	650	90	-40
21	15	3950	3820	1580	565	60	1900	755	180	-45
22	15.5	4200								
23										
24										
25										

Inc. No.	Load per Jack	Strain Gage Number								
		14	17	18	21	22				
1	.148	0	0	0	0	0				
2	1	0	-10	- 5	-10	- 5				
3	2	0	-15	-10	-15	- 5				
4	3	0	-25	-10	-25	-10				
5	4	0	-35	-20	-40	-15				
6	5	0	-45	-25	-55	-20				
7	6	0	-55	-30	-70	-20				
8	7	5	-65	-35	-85	-25				
9	7.5	5	-75	-40	-90	-25				
10	8.0	5	-85	-40	-100	-30				
11	8.5	5	-95	-35	-110	-30				
12	9.0	5	-100	-30	-120	-30				
13	9.5	10	-90	- 5	-120	-35				
14	10	10	-90	5	-125	-40				
16	11	5	-50	125	-90	-55				
18	12	5	-45	225	-60	-65				
19	13		-30	315	- 5	-40				
20	14		-20	455	45	85				
21	15		- 5	535	75	150				
22	15.5									
23										
24										
25										

TABLE B.1.2 CENTERLINE STRAIN DISTRIBUTION AND BEAM DEFLECTION

Inc.	Strain (in/in x 10^{-5})							
	1	2	3	5	6	7	8	9
i	0	0	0	0	0	0	0	0
ii	63	52	31	11	- 1	10	- 6	- 1
1	158	127	90	40	-11	30	36	37
2	156	126	88	41	- 9	32	37	38
3	150	121	87	41	2	35	40	39
4	144	116	84	45	7	39	41	40
5	138	112	83	46	17	42	44	43
6	134	108	79	50	24	47	49	46
7	124	102	75	49	28	51	49	48
8	122	99	78	54	34	55	55	52
9	119	96	73	53	38	57	57	54
10	116	94	72	52	40	59	56	53
11	113	91	71	54	43	61	61	58
12	109	89	68	53	47	62	63	59
13	106	82	65	57	47	66	65	61
14	107	81	64	56	51	68	68	64
16	114	63	63	58	62	80	77	73
18	114	75	58	60	71	85	84	81
19	116	64	53	41	87	87	93	92
20	-54	-51	-21	27	102	105	114	111
21	-136	-107	-57	18	111	114	125	120
22								

(i) Before Release ; (ii) After Release ; (-) Tension

Inc.	Load (kips)	Deflection (in)		
		North	℄	South
1	0.15	0	0	0
2	1	0.01	0.02	0.02
3	2	0.04	0.05	0.04
4	3	0.07	0.09	0.06
5	4	0.10	0.12	0.10
6	5	0.13	0.16	0.13
7	6	0.16	0.20	0.16
8	7	0.19	0.23	0.20
9	7.5	0.21	0.25	0.22
10	8	0.23	0.28	0.23
11	8.5	0.26	0.31	0.26
12	9	0.29	0.35	0.30
13	9.5	0.33	0.39	0.33
14	10	0.36	0.43	0.36
16	11	0.54	0.66	0.56
18	12	0.73	0.84	0.73
19	13	0.94	1.16	0.95
20	14	1.24	1.55	1.25
21	15	1.74	1.82	1.51
22	15.5		2.07	

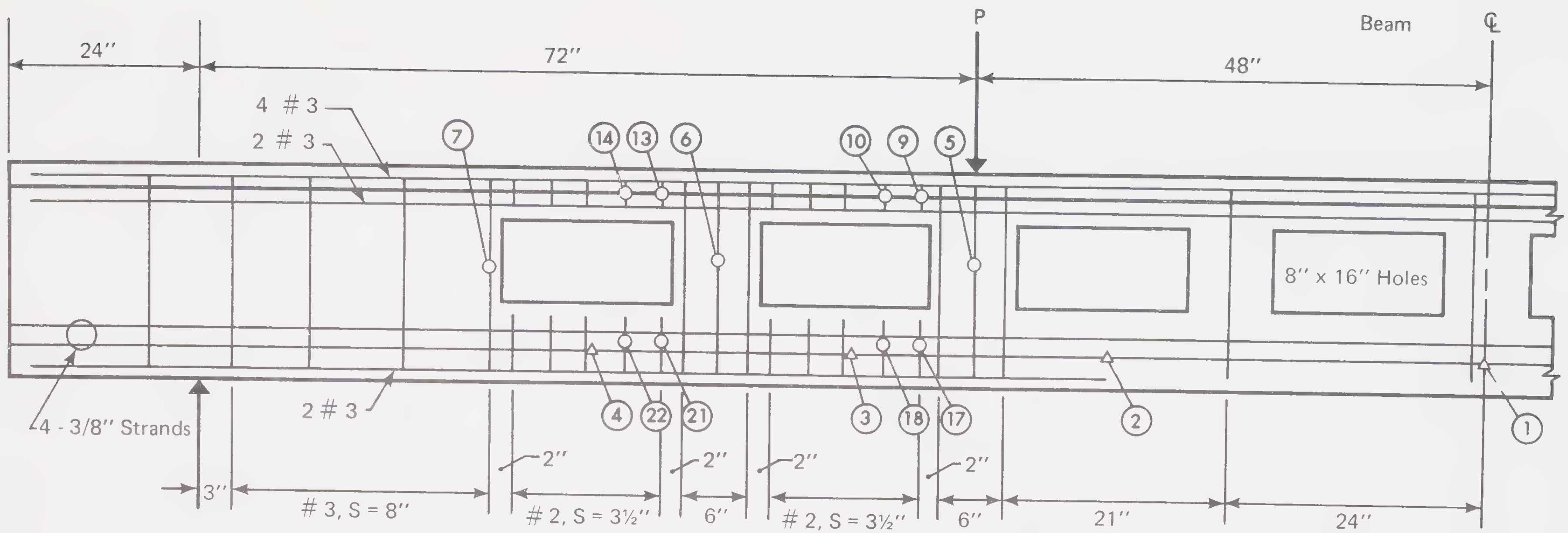


FIGURE B.1. Reinforcement Details and Strain Gage Locations for Beam 1–16–6

TABLE B.2.1 ELECTRICAL STRAIN GAGE MEASUREMENTS (MICRO INCHES PER INCH)

BEAM NO. 2-16-4

Inc. No.	Load per Jack	Strain Gage Number								
		1	2	4	6	7	14	21	22	
1	0.148	0	0	0	0	0	0	0	0	
3	2	30	25	20	- 5	0	-10	-15	-15	
5	4	65	55	45	-15	- 5	-30	-25	-30	
6	5	80	75	60	-20	- 5	-25	-35	-35	
8	7	115	105	75	-25	-25	-30	-45	-50	
10	9	155	140	95	-25	-25	-35	-65	-65	
12	11	200	185	110	-15	70	-50	-50	-55	
14	13	235	225	135	- 5	175	-60	90	10	
16	14	270	255	145	5	215	-65	175	80	
18	15	330	290	155	15	270	-60	235	140	
20	16	470	355	175	30	355	-30	340	245	
22	17	910	480	225	35	435	35	365	280	
24	18	1230	615	355	45	550	165	380	305	
26	19	1560	785	550	40	720	610	405	330	
28	20	1845	910	950	40	920	1060	420	350	
30	21	2270	1080	1155	870	1245	1440	485	380	
32	22	2430	1140	1285	925	1380	1670	530	400	
34	23.5	2850	1240	1555	980	1700	2270	550	430	
35	24	3035	1300	1770	1000	1900		525	450	
36	24.5									

TABLE B.2.2 CENTERLINE STRAIN DISTRIBUTION AND BEAM DEFLECTION

Inc.	Strain (in/in x 10^5)							
	1	2	3	5	6	7	8	9
i	0	0	0	0	0	0	0	0
ii	58	46	36	4	- 1	1	0	3
1	139	117	92	40	28	32	35	25
3	123	113	89	43	31	33	37	35
5	125	108	86	44	37	36	35	37
6	121	106	84	43	36	38	41	39
8	112	103	84	50	42	41	42	39
10	107	103	78	50	46	46	50	45
12	97	90	73	59	49	50	55	53
14	92	36	70	49	54	54	57	54
16	83	80	68	41	53	58	59	61
18	62	71	65	54	61	60	64	61
20	26	47	55	54	62	66	68	65
22	-60	- 6	33	55	66	69	73	71
24	-147	-54	15	51	71	69	73	75
26	-259	-63	1	43	85	81	88	89
28	-326	-78	-10	38	93	92	93	98
30	-469	-110	-29	27	100	97	102	103
32	-514	-121	-37	25	104	102	108	110
34	-600	-237	-42	19	111	105	115	119
35								
36								

(i) Before Release ; (ii) After Release ; (-) Tension

Inc.	Load (kips)	Deflection (in)		
		North	℄	South
1	0.15	0.00	0.00	0.00
3	2	.02	.02	.02
5	4	.04	.05	.05
6	5	.07	.07	.07
8	7	.11	.11	.10
10	9	.15	.16	.14
12	11	.20	.22	.20
14	13	.26	.28	.25
16	14	.31	.33	.30
18	15	.34	.38	.33
20	16	.42	.46	.41
22	17	.53	.60	.52
24	18	.73	.85	.72
26	19	.93	1.10	.92
28	20	1.07	1.27	1.05
30	21	1.27	1.52	1.25
32	22	1.37	1.63	1.35
34	23.5	1.57	1.86	1.54
35	24	1.67	2.02	1.66
36	24.5		2.40	

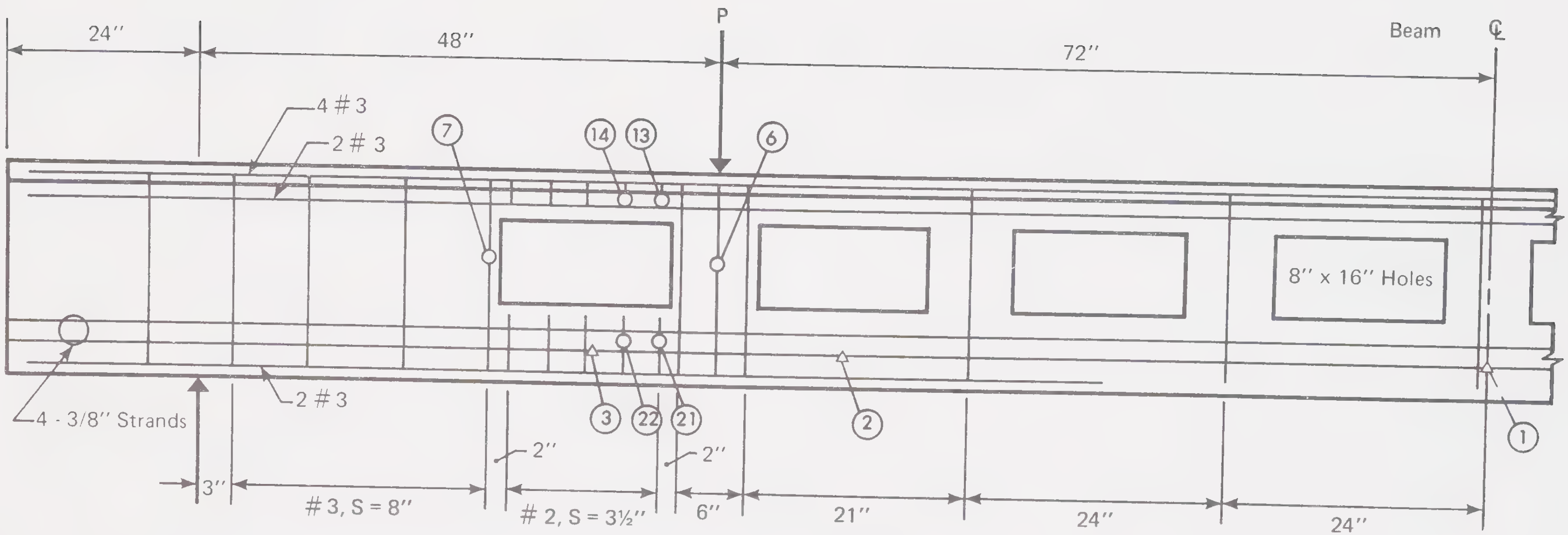


FIGURE B.2. Reinforcement Details and Strain Gage Locations for Beam 2–16–4

TABLE B.3.1 ELECTRICAL STRAIN GAGE MEASUREMENTS (MICRO INCHES PER INCH)

BEAM NO. 3-16-6

Inc. No.	Load per Jack	Strain Gage Number								
		1	2	3	4	5	6	7	9	10
1	.148	0	0	0	0	0	0	0	0	0
2	2	50	50	40	25	- 5	10	15	15	10
3	4	110	105	80	65	-10	50	45	35	10
4	5	140	135	100	80	-15	60	55	60	5
5	6	175	165	115	95	-15	80	65	90	5
6	7	210	195	135	115	-20	110	80	90	5
7	8	240	230	155	125	-15	760	240	85	5
8	9	290	270	170	150	- 5	920	365	90	5
9	10	325	310	195	160	0	1040	485	90	5
10	10.5	360	340	215	165	10	1130	560	100	5
11	11	420	375	235	170	20	1220	610	115	5
12	11.5	465	405	250	175	35	1285	660	130	0
13	12	540	440	270	180	45	1350	720	150	0
14	12.5	715	470	290	185	55	1410	775	170	0
15	13	885	800	310	185	70	1470	865	200	0
16	14	1280	1610	500	200	95	1610	1080	315	20
17	15	1565	1735	665	210	700	1730	1230	375	25
18	16	2040	2080	875	260	760	2030	1410	570	90
19	17	2400								

Inc. No.	Load per Jack	Strain Gage Number								
		13	14	17	18	21	22			
1	.148	0	0	0	0	0	0			
2	2	20	5	-10	- 5	-10	- 5			
3	4	20	0	-30	-15	-35	-15			
4	5	25	0	-40	-20	-50	-20			
5	6	30	0	-50	-20	-65	-30			
6	7	35	0	-60	-30	-80	-30			
7	8	45	0	-75	-25	-90	-40			
8	9	55	- 5	-75	- 5	-110	-45			
9	10	70	- 5	-55	10	-125	-50			
10	10.5	80	- 5	-60	25	-135	-55			
11	11	95	- 5	-70	50	-140	-60			
12	11.5	105	- 5	-75	65	-145	-65			
13	12	120	- 5	-80	95	-150	-65			
14	12.5	130	- 5	-85	125	-135	-55			
15	13	150	- 5	-80	170	-115	-45			
16	14	225	0	-55	270	-90	5			
17	15	300	15	-60	345	-105	50			
18	16	380	40	-45	435	-120	90			
19	17									

TABLE B.3.2 CENTERLINE STRAIN DISTRIBUTION AND BEAM DEFLECTION

Inc.	Strain (micro in/in)							
	1	2	3	5	6	7	8	9
i	0	0	0	0	0	0	0	0
ii	89	75	57	17	3	7	4	4
1	195	162	122	52	32	39	38	34
2	191	159	121	54	37	39	43	40
3	179	146	115	53	42	50	49	46
4	173	144	113	58	44	53	52	49
5	167	140	111	62	50	58	55	53
6	161	134	108	62	48	57	60	57
7	156	131	105	64	56	61	60	60
8	147	125	102	69	61	68	66	65
9	139	121	98	69	64	71	70	68
10	130	114	95	69	69	75	77	70
11	114	107	91	70	69	78	77	74
12	101	97	88	70	70	77	76	72
13	83	88	83	72	76	84	82	78
14	65	74	78	72	76	84	83	81
15	21	47	66	71	83	89	85	86
16	-72	- 7	35	65	81	99	98	98
17	-135	-47	13	63	99	105	103	106
18	-280	-193	-19	54	113	116	115	118
19								
(i) Before Release ; (ii) After Release ; (-) Tension								

Inc.	Load (kips)	Deflection (in)		
		North	℄	South
1	0.15	0.00	0.00	0.00
2	2	.04	.05	.04
3	4	.10	.12	.10
4	5	.14	.15	.13
5	6	.17	.19	.16
6	7	.20	.23	.19
7	8	.24	.27	.22
8	9	.29	.33	.28
9	10	.34	.38	.32
10	10.5	.37	.42	.36
11	11	.41	.47	.40
12	11.5	.44	.50	.43
13	12	.48	.54	.46
14	12.5	.52	.59	.50
15	13	.61	.69	.59
16	14	.84	.99	.81
17	15	.99	1.18	.97
18	16	1.26	1.51	1.22
19	17		1.73	

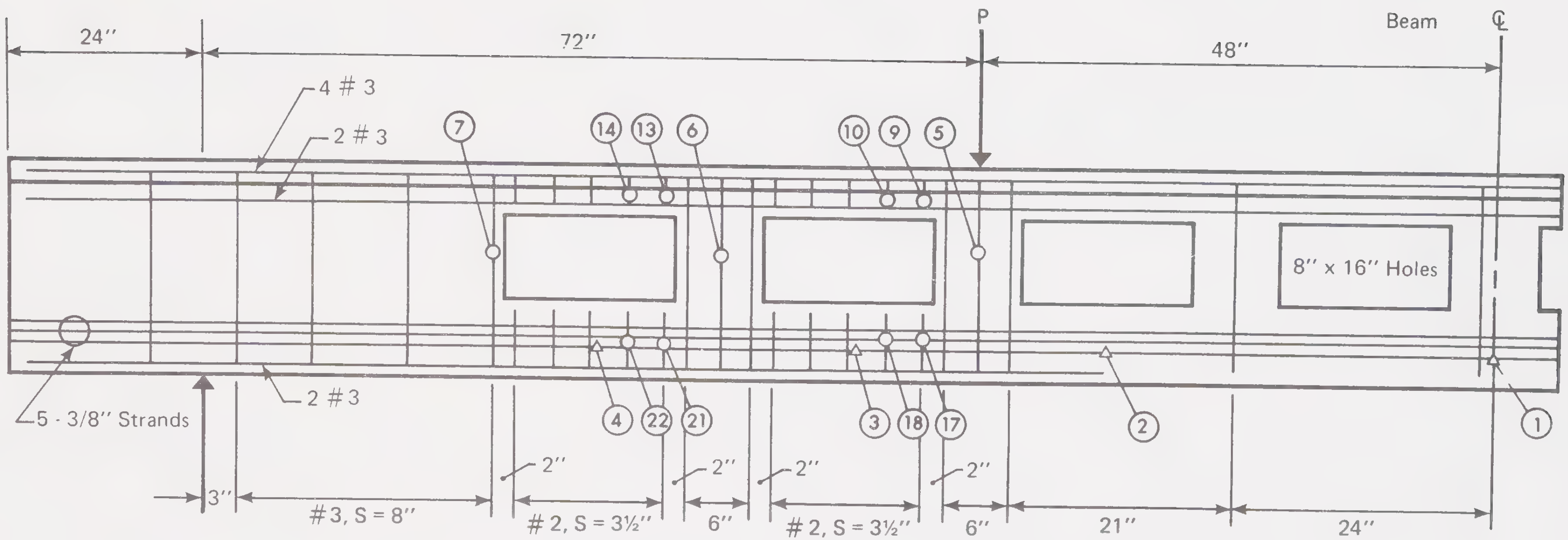


FIGURE B.3. Reinforcement Details and Strain Gage Locations for Beam 3–16–6

TABLE B.4.1 ELECTRICAL STRAIN GAGE MEASUREMENTS (MICRO INCHES PER INCH)

BEAM NO. 4-16-6

Inc. No.	Load per Jack	Strain Gage Number									
		1	3	4	5	6	7	9	10	13	14
1	.148	0	0	0	0	0	0	0	0	0	0
3	1.0	20	15	10	- 5	0	0	0	0	0	0
5	2.0	50	40	25	-10	5	0	10	0	10	0
7	3.0	80	60	40	-15	20	0	20	0	15	0
9	4.0	110	90	65	-15	35	0	30	0	25	0
10	4.5	130	100	75	-15	40	0	35	0	25	0
11	5.0	150	115	90	-15	50	- 5	45	0	25	0
12	5.5	165	135	100	-15	60	-15	70	0	30	0
13	6.0	190	160	120	-10	75	-20	80	0	30	0
14	6.5	200	195	135	- 5	110	-25	85	0	30	0
15	7.0	220	230	185	0	770	-35	140	- 5	30	5
16	7.5	240	300	215	0	850	-40	160	- 5	40	5
17	8.0	255	380	240	0	920	-40	280	- 5	45	5
19	9.0	315	685	345	10	1120	-55	245	-10	60	10
21	10.0	420	970	485	25	1260	-70	345	-10	85	10
22	10.5	640	1180	610	50	1320	-80	420	-10	110	20
23	11.0	1065	1460	770	435	1390	-85	540	-10	150	25
24	11.5	1195	2200	2380	950		-135	730	515	120	80
25	12.0	1395	2650	2780	1070		-135	790	480	105	85
26	12.5	1550	3175	3400				910			

TABLE B.4.2 CENTERLINE STRAIN DISTRIBUTION AND BEAM DEFLECTION

Inc.	Strain (in/in x 10^5)							
	1	2	3	5	6	7	8	9
i	0	0	0	0	0	0	0	0
ii	72	53	36	10	2	7	9	6
1	157	134	96	46	40	40	39	21
3	160	135	99	50	45	44	42	25
5	154	131	95	52	47	48	45	27
7	149	126	92	51	49	50	46	35
9	143	123	88	58	54	57	55	41
10	140	116	88	60	57	59	58	41
11	136	118	87	60	57	58	57	46
12	131	115	87	62	57	60	59	49
13	129	101	86	61	59	63	62	52
14	125	109	85	63	62	64	63	51
15	120	108	89	62	65	69	67	56
16	119	105	86	59	64	68	68	59
17	114	103	87	61	68	69	72	62
19	99	93	83	61	72	75	74	67
21	70	74	74	63	77	79	80	72
22	30	52	63	65	81	83	83	76
23	-48	- 1	38	59	89	88	90	88
24	-56	-19	23	55	99	97	98	95
25								
26								

(i) Before Release ; (ii) After Release ; (-) Tension

Inc.	Load (kips)	Deflection (in)		
		North	℄	South
1	0.15	0.00	0.00	0.00
3	1	.02	.03	.02
5	2	.05	.05	.05
7	3	.08	.09	.08
9	4	.12	.14	.11
10	4.5	.14	.16	.13
11	5	.15	.18	.15
12	5.5	.18	.20	.17
13	6	.20	.23	.19
14	6.5	.22	.25	.21
15	7	.25	.28	.24
16	7.5	.28	.31	.26
17	8	.31	.34	.28
19	9	.40	.44	.36
21	10	.49	.53	.44
22	10.5	.58	.64	.52
23	11	1.35	1.29	.95
24	11.5	1.53	1.46	1.09
25	12	1.97	1.86	1.36
26	12.5		1.92	

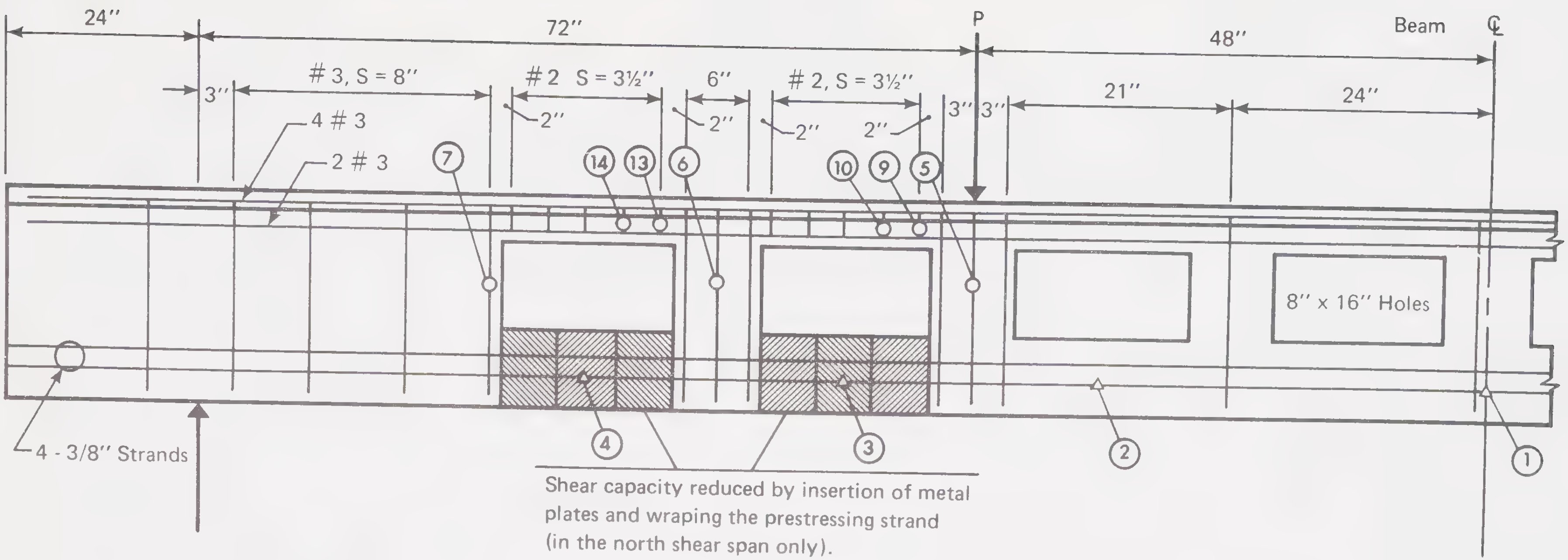


FIGURE B.4. Reinforcement Details and Strain Gage Locations for Beam 4–16–6

TABLE B.5.1 ELECTRICAL STRAIN GAGE MEASUREMENTS (MICRO INCHES PER INCH)

BEAM NO. 5-16-4

Inc. No.	Load per Jack	Strain Gage Number								
		1	2	4	6	7	13	14		
1	.148	0	0	0	0	0	0	0		
2	1.0	15	10	15	5	5	5	0		
3	2.0	30	30	30	5	10	10	0		
4	4.0	80	105	85	65	70	40	40		
5	6.0	125	145	125	65	110	50	40		
6	7.0	145	160	145	65	150	45	35		
7	8.0	165	180	170	70	170	50	40		
8	9.0	195	200	225	70	155	35	35		
9	10.0	220	225	295	70	140	25	35		
10	11.0	250	250	450	80	115	20	35		
11	12.0	270	275	600	90	100	30	40		
12	13.0	300	300	770	95	80	50	45		
13	14.0	340	340	1030	110	55	120	60		
14	15.0	400	375	1230	130	155	250	75		
16	16.0	500	430	1485	165	700	570	145		
17	17.0	760	480	1755	185	740	750	370		
19	18.0	1065	1445	2030	865	770	835	500		
21	19.0	1635	1890	2515	900	850	945	790		
23	20.0	1940	2180	2885	910	930	780	1370		
24	20.5	2185	2400	3095	970	1080	730	1590		

TABLE B.5.2 CENTERLINE STRAIN DISTRIBUTION AND BEAM DEFLECTIONS

Inc.	Strain (in/in x 10^5)							
	1	2	3	5	6	7	8	9
i	0	0	0	0	0	0	0	0
ii	67	59	36	4	2	2	3	2
1	137	124	92	34	23	28	30	28
2	138	123	94	31	19	28	30	27
3	136	123	91	33	16	29	33	28
4	128	116	89	37	25	34	36	35
5	119	111	86	36	31	39	40	39
6	117	108	85	37	29	40	41	40
7	113	105	82	41	30	39	44	42
8	108	104	82	41	37	45	49	48
9	105	100	78	43	36	43	49	47
10	100	98	77	45	42	48	52	51
11	95	94	75	44	44	51	54	52
12	92	89	71	45	47	53	57	55
13	83	83	70	46	48	55	58	58
14	67	75	64	46	52	58	63	60
16	43	59	56	47	55	60	66	65
17	-14	11	39	49	58	64	70	69
19	-68	- 6	21	42	66	71	77	77
21	-139	-90	-14	38	73	76	84	89
23	-183	-130	-24	37	82	84	95	94
24	-219	-168	-148	35	87	88	98	98

(i) Before Release; (ii) After Release ; (-) Tension

Inc.	Load (kips)	Deflection (in)		
		North	℄	South
1	0.15	0.00	0.00	0.00
2	1	.02	.02	.01
3	2	.03	.03	.02
4	4	.07	.08	.06
5	6	.12	.13	.11
6	7	.14	.15	.12
7	8	.16	.17	.14
8	9	.19	.20	.17
9	10	.21	.23	.20
10	11	.25	.27	.23
11	12	.28	.31	.26
12	13	.32	.35	.29
13	14	.37	.40	.34
14	15	.43	.46	.39
16	16	.52	.56	.48
17	17	.65	.71	.60
19	18	.89	.99	.82
21	19	1.17	1.32	1.10
23	20	1.41	1.58	1.31
24	20.5	1.61	1.79	1.48

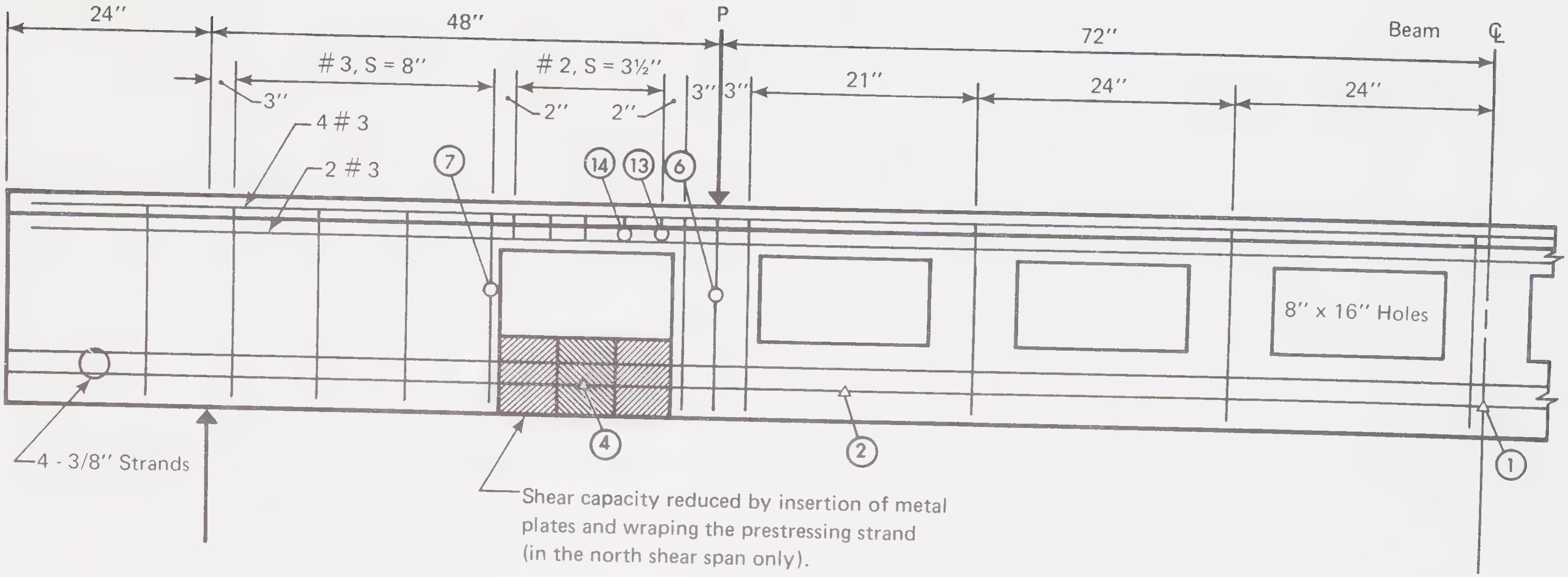


FIGURE B.5. Reinforcement Details and Strain Gage Locations for Beam 5–16–4

TABLE B.6.1 ELECTRICAL STRAIN GAGE MEASUREMENTS (MICRO INCHES PER INCH)

BEAM NO. 6-16-6

Inc. No.	Load per Jack	Strain Gage Number								
		1	2	4	5	6	7	9	10	13
1	.148	0	0	0	0	0	0	0	0	0
2	2	45	50	25	-10	5	20	20	0	5
3	4	110	115	60	-20	45	75	85	- 5	20
4	6	180	185	95	-30	90	180	130	- 5	30
5	7	210	220	110	-30	460	300	170	- 5	35
6	7.5	230	235	115	-30	490	325	170	-10	35
7	8	245	255	115	-30	540	325	175	-10	45
8	8.5	275	290	120	-30	620	370	185	-10	55
9	9	315	320	120	-25	670	450	195	-10	70
10	9.5	355	350	125	-20	710	530	215	-10	75
11	10	400	385	130	-20	750	580	240	-10	85
12	10.5	600	450	130	-10	805	660	540	-10	110
13	11	1130	820	140	0	855	730	1030	- 5	135
15	12	1420	1120	155	25	935	830	890	0	180
16	13	1895	1570	175	55	1010	940	1530	10	250
17	14	2570	2240	235	370	1125	1120	2910	30	355
18	15	3030	3220	340	390	1170	1250		150	470
19	16	3670	4000	510	430	1265	1490		730	860
20	17	4120	4565	650	460	1375	1650		970	1370
21	17.5		4900							

Inc. No.	Load per Jack	Strain Gage Number				
		14	17	18	21	22
1	.148	0	0	0	0	0
2	2	0	-15	-10	-10	-10
3	4	0	-30	-20	-25	-20
4	6	0	-40	-25	-45	-30
5	7	0	-15	-20	-50	-35
6	7.5	0	0	-20	-60	-35
7	8	0	25	-20	-60	-40
8	8.5	0	60	-20	-70	-45
9	9	0	110	-25	-80	-50
10	9.5	0	130	-30	-70	-55
11	10	0	150	-30	-65	-60
12	10.5	0	205	-10	-40	-70
13	11	5	245	15	-10	-50
15	12	10	300	65	- 5	-15
16	13	30	345	110	20	45
17	14	65	405	205	100	180
18	15	135	415	260	155	390
19	16	385	540	275	210	505
20	17	480	560	290	230	550
21	17.5					

TABLE B.6.2 CENTERLINE STRAIN DISTRIBUTION AND BEAM DEFLECTIONS

Inc.	Strain (in/in x 10^5)							
	1	2	3	5	6	7	8	9
i	0	0	0	0	0	0	0	0
ii	80	64	42	11	2	0	3	- 3
1	172	144	10	47	23	29	9	1
2	166	140	107	47	26	33	35	12
3	154	130	102	50	33	42	43	28
4	139	123	98	51	42	50	54	44
5	133	119	94	54	45	52	55	48
6	130	114	92	54	47	55	59	51
7	126	110	91	54	48	54	58	51
8	119	109	89	53	51	57	63	57
9	106	101	88	54	54	62	66	60
10	96	96	85	56	57	62	66	62
11	80	89	79	54	58	65	71	65
12	41	65	65	55	63	69	74	68
13	-40	9	32	51	69	75	81	75
15	-91	-35	3	42	81	86	90	88
16	-190	-109	-35	42	92	95	101	99
17	-348	-246	-109	41	108	110	115	114
18	-464	-358	-176	42	119	123	129	127
19	-626	-509	-268	38	133	135	142	142
20	-754	-601	-316	37	143	145	152	153
21								

(i) Before Release; (ii) After Release ; (-) Tension

Inc.	Load (kips)	Deflection (in)		
		North	℄	South
1	0.15	0.00	0.00	0.00
2	2	.04	.05	.05
3	4	.11	.11	.11
4	6	.17	.19	.16
5	7	.21	.23	.20
6	7.5	.23	.26	.22
7	8	.25	.28	.24
8	8.5	.28	.31	.27
9	9	.31	.34	.30
10	9.5	.34	.38	.32
11	10	.37	.42	.35
12	10.5	.44	.48	.42
13	11	.58	.66	.56
15	12	.77	.91	.75
16	13	.97	1.17	.94
17	14	1.27	1.55	1.26
18	15	1.53	1.87	1.50
19	16	1.89	2.31	1.82
20	17	2.10	2.54	2.01
21	17.5		2.89	

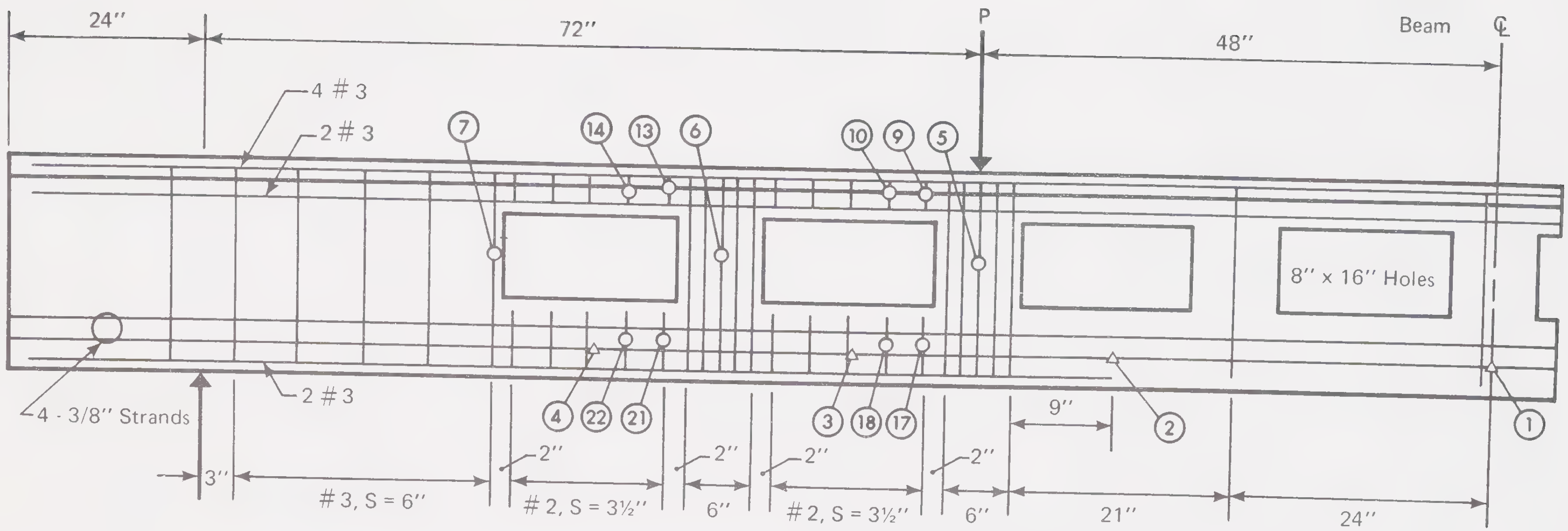


FIGURE B.6. Reinforcement Details and Strain Gage Locations for Beam 6–16–6

TABLE B.7.1 ELECTRICAL STRAIN GAGE MEASUREMENTS (MICRO INCHES PER INCH)

BEAM NO. 7-16-6

Inc. No.	Load per Jack	Strain Gage Number								
		1	2	3	4	5	6	7	10	13
1	.148	0	0	0	0	0	0	0	0	0
2	2	40	45	35	20	-10	5	0	0	15
3	4	100	110	75	55	-15	35	50	0	30
4	6	160	170	110	85	-15	410	350	- 5	50
5	7	185	200	120	100	-10	510	460	- 5	60
6	8	215	230	140	115	-10	615	565	-10	70
7	9	260	270	170	130	-10	730	695	-10	70
8	10	290	320	200	140	- 5	810	790	-15	75
9	11	370	395	235	140	5	925	835	-15	95
10	12	460	460	265	145	20	995	885	-20	110
11	13	770	550	300	155	30	1060	970	-15	145
12	14	1250	1090	440	170	30	1120	1090	-10	190
13	15	1560	1500	630	170	370	1170	1175	0	230
14	16	2020	1990	860	200	405	1245	1285	60	330
15	17	2420	2360	1010	220	430	1300	1350	200	440
16	18	3110	2810	1245	325	450	1375	1460	390	510
17	19						1450			

Inc. No.	Load per Jack	Strain Gage Number								
		14	17	18	21	22				
1	.148	0	0	0	0	0				
2	2	0	-10	-10	- 5	- 5				
3	4	- 5	-15	-20	-20	-15				
4	6	- 5	-25	-35	-40	-25				
5	7	- 5	-20	-40	-50	-30				
6	8	-10	-15	-20	-60	-35				
7	9	- 5	20	15	-70	-40				
8	10	-10	40	55	-95	-45				
9	11	-10	80	90	-80	-55				
10	12	-10	115	120	-50	-40				
11	13	-10	165	160	-25	-30				
12	14	0	235	260	25	20				
13	15	20	270	310	50	50				
14	16	100	330	390	90	90				
15	17	230	370	450	120	255				
16	18	440	460	500	150	380				
17	19									

TABLE B.7.2 CENTERLINE STRAIN DISTRIBUTION AND BEAM DEFLECTIONS

Inc.	Strain (in/in x 10^5)							
	1	2	3	5	6	7	8	9
i	0	0	0	0	0	0	0	0
ii	107	89	61	10	4	- 1	3	- 1
1	203	169	23	54	43	40	50	35
2	198	168	123	57	47	43	55	40
3	188	161	118	60	56	48	60	47
4	175	150	111	64	61	56	67	54
5	170	147	110	63	66	60	72	57
6	163	142	108	69	70	63	74	62
7	156	137	109	66	74	67	79	64
8	148	130	102	67	76	70	82	68
9	27	116	93	69	84	77	88	74
10	96	101	90	70	87	78	92	78
11	20	62	73	71	90	84	95	83
12	-32	20	47	74	105	95	107	97
13	-66	- 3	34	56	114	105	117	107
14	-219	-43	12	48	126	117	129	119
15	-264	-81	- 9	35	135	126	139	129
16	-323	-133	-21	29	147	138	151	139
17								

(i) Before Release; (ii) After Release; (-) Tension

Inc.	Load (kips)	Deflection (in)		
		North	℄	South
1	0.15	0.00	0.00	0.00
2	2	.04	.05	.04
3	4	.10	.11	.09
4	6	.17	.19	.16
5	7	.20	.23	.19
6	8	.23	.26	.22
7	9	.28	.32	.26
8	10	.32	.36	.31
9	11	.39	.44	.38
10	12	.45	.52	.45
11	13	.57	.67	.56
12	14	.81	.97	.80
13	15	.96	1.17	.96
14	16	1.18	1.45	1.18
15	17	1.37	1.68	1.37
16	18	1.61	1.98	1.61
17	19		2.33	

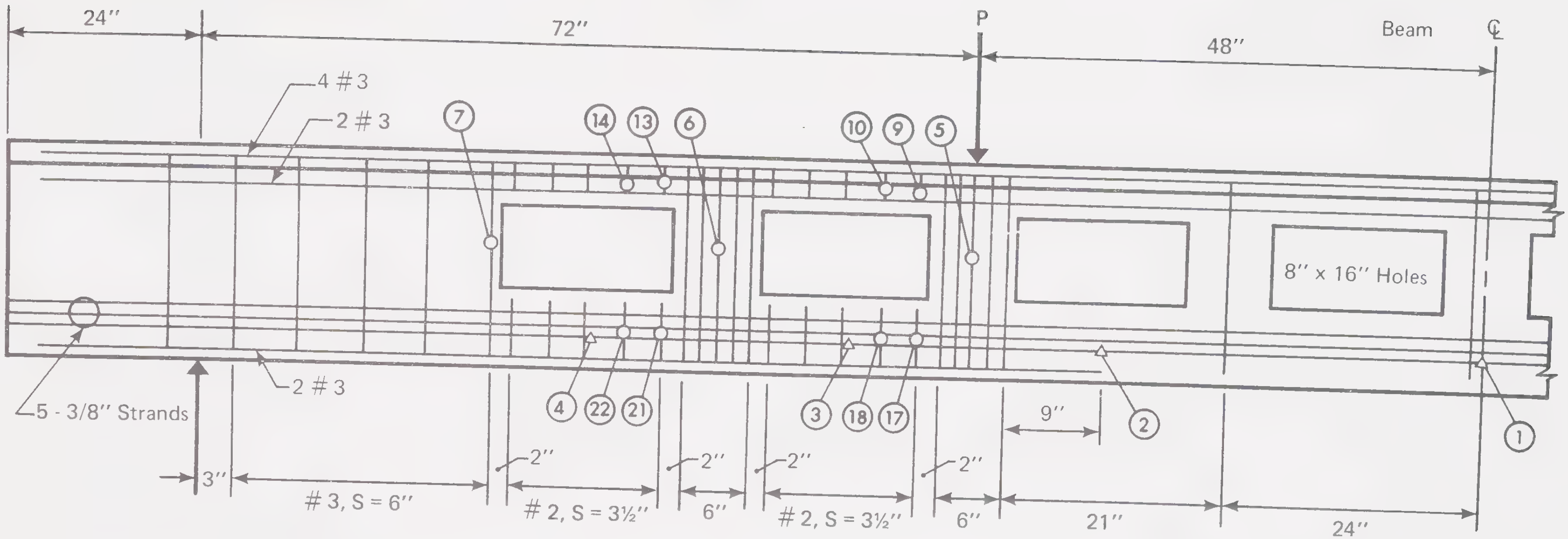


FIGURE B.7. Reinforcement Details and Strain Gage Locations for Beam 7–16–6

TABLE B.8.1 ELECTRICAL STRAIN GAGE MEASUREMENTS (MICRO INCHES PER INCH)

BEAM NO. 8-16-7L

Inc. No.	Load per Jack	Strain Gage Numbers								
		1	1A	2	3	4	5	6	7	9
1	0.120	0	0	0	0	0	0	0	0	0
2	.25	10	10	10	5	5	0	0	0	0
3	.5	35	35	30	25	15	- 5	0	10	0
4	.75	50	55	50	35	25	- 5	0	15	5
5	1.0	70	75	65	50	30	- 5	0	25	5
6	1.25	95	100	90	65	45	0	10	40	10
7	1.5	125	135	110	80	60	10	30	65	20
8	1.75	150	160	140	95	70	15	45	60	35
9	2.0	180	190	170	110	80	25	60	55	55
10	2.25	220	235	195	130	90	35	75	95	60
11	2.50	290	290	245	150	100	40	110	155	60
12	2.75	415	335	270	155	110	45	880	235	75
13	3.0	690	390	315	170	120	50	975	260	75
15	3.5	1390	1215	580	230	130	80	1180	350	70
17	4.0	2195	2405	1060	340	145	1045	1380	440	80
19	4.5	3240	3720	1590	640	175	1190	1530	610	110
21	5.0	6400	7500	2340	930	370	1340	1750	795	170
22	5.25	7180	8560	2440	950	400	1405	1845	875	180
23	5.5		11850	2690	1030	490	1480	1940	1010	220
24	5.75		30500	3030	1170	600	1540	2025	1090	275
Inc. No.	Load per Jack	Strain Gage Numbers								
		10	13	14	17	18	21	22		
1	0.120	0	0	0	0	0	0	0		
2	.25	- 5	-10	- 5	- 5	- 5	0	0		
3	.5	- 5	-15	-10	-15	-10	10	0		
4	.75	- 5	-20	-10	-20	-15	25	0		
5	1.0	- 5	-30	-15	-25	-15	40	0		
6	1.25	-10	-40	-20	-30	-20	60	0		
7	1.5	-10	-50	-25	-40	-25	50	0		
8	1.75	-10	-60	-30	-45	-25	40	0		
9	2.0	- 5	-70	-30	-50	-30	35	0		
10	2.25	- 5	-85	-35	-65	-30	25	0		
11	2.50	- 5	-100	-40	-80	-30	20	0		
12	2.75	-10	-105	-45	-100	- 5	15	5		
13	3.0	-10	-105	-45	-110	20	15	5		
15	3.5	-10	-70	-40	-55	30	20	5		
17	4.0	-10	- 5	-30	20	80	30	5		
19	4.5	-10	0	-15	60	140	60	5		
21	5.0	-10	60	290	110	210	205	25		
22	5.25	-10	60	310	130	225	305	35		
23	5.5	-10	75	375	205	235	420	60		
24	5.75	-10	90	450	250	250	475	75		

TABLE B.8.2 CENTERLINE STRAIN DISTRIBUTION AND BEAM DEFLECTIONS

Inc.	Strain (in/in x 10^5)							
	1	2	3	5	6	7	8	9
i	0	0	0	0	0	0	0	0
ii	60	47	32	5	0	2	2	1
1	156	126	93	50	37	29	34	33
2	156	124	93	52	39	30	36	36
3	150	120	91	51	43	33	37	39
4	147	118	90	51	43	35	41	40
5	144	115	87	52	46	36	42	44
6	136	110	84	52	49	40	45	45
7	132	107	83	52	53	45	49	48
8	125	104	81	54	55	46	53	51
9	121	101	80	55	56	49	54	54
10	113	94	74	56	60	53	59	58
11	85	80	72	55	65	47	63	61
12	51	60	63	55	70	53	69	67
13	2	33	52	51	76	69	76	74
15	-110	-55	- 2	35	97	90	96	93
17	-249	-153	-67	-10	117	110	118	106
19	-372	-262	-149	-45	140	136	143	139
21	-693	-595	-406	-146	190	186	193	188
22	-808	-691	-482	-169	204	200	207	205
23	-1712	-981	-712	-233	236	233	241	239
24			-1491	-926	309	310	321	318

(i) Before Release; (ii) After Release; (-) Tension

Inc.	Load (kips)	Deflection (in)		
		North	℄	South
1	0.12	0.00	0.00	0.00
2	.25	.01	.01	.01
3	.50	.03	.04	.03
4	.75	.05	.06	.05
5	1.00	.06	.08	.07
6	1.25	.09	.11	.09
7	1.50	.11	.14	.12
8	1.75	.14	.17	.14
9	2.00	.16	.20	.17
10	2.25	.20	.24	.20
11	2.50	.24	.30	.25
12	2.75	.29	.36	.29
13	3.00	.35	.45	.35
15	3.50	.59	.78	.60
17	4.00	.87	1.15	.87
19	4.50	1.19	1.58	1.20
21	5.00	1.81	2.61	1.91
22	5.25	2.08	2.87	2.09
23	5.50	2.54	3.55	2.55
24	5.75	4.08	5.90	4.08

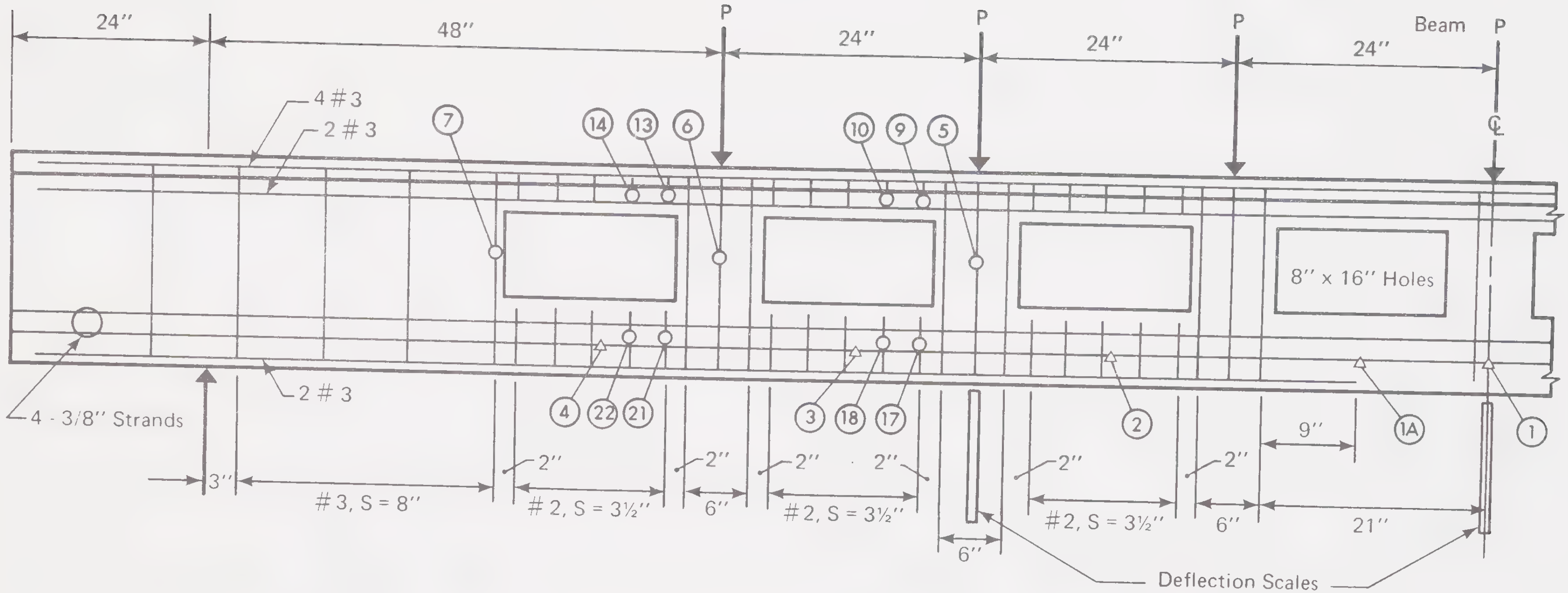


FIGURE B.8. Reinforcement Details and Strain Gage Locations for Beam 8–16–7L

TABLE B.9.1 ELECTRICAL STRAIN GAGE MEASUREMENTS (MICRO INCHES PER INCH)

BEAM NO. 9-16-7L

Inc. No.	Load per Jack	Strain Gage Numbers								
		1	1A	3	5	6	7	9	10	13
1	0.12	0	0	0	0	0	0	0	0	0
2	0.5	35	40	30	105	5	0	10	5	5
3	1.0	75	85	55	170	20	0	25	5	5
4	1.5	130	145	90	1370	35	30	20	10	10
5	2.0	190	210	125	2700	60	170	20	10	10
6	2.25	225	250	130	2700	890	390	30	10	5
7	2.5	270	305	165	-1800	1080	475	30	10	5
8	2.75	305	340	185	-1800	1220	500	30	10	5
9	3.0	365	385	200		1380	535	30	10	5
10	3.25	440	450	220		1625	610	30	15	10
11	3.5	580	1320	245		1735	700	30	15	20
12	3.75	920	1715	275		1825	885	35	15	45
13	4.0	1320	1990	315		1940	1020	40	20	60
14	4.5	2060	2140	405		2180	1230	90	20	150
15	5.0	3430	3120	695			1530	215	30	365
16	5.5	4370								

Inc. No.	Load per Jack	Strain Gage Numbers								
		14	17	18	21	22				
1	0.12	0	0	0	0	0				
2	0.5	0	-10	-10	-15	- 5				
3	1.0	5	-20	-15	-30	-10				
4	1.5	5	-40	-25	-60	-25				
5	2.0	10	-60	-30	-80	-35				
6	2.25	10	-80	-30	-100	-45				
7	2.5	10	-85	-30	-110	-50				
8	2.75	10	-85	-30	-130	-55				
9	3.0	10	-80	-20	-145	-60				
10	3.25	10	-80	-10	-170	-70				
11	3.5	15	-80	10	-190	-70				
12	3.75	15	-65	60	-210	-75				
13	4.0	20	-50	125	-230	-80				
14	4.5	25	-10	270	-225	-80				
15	5.0	55	35	350	-100	230				
16	5.5									

TABLE B.9.2 CENTERLINE STRAIN DISTRIBUTION AND BEAM DEFLECTIONS

Inc.	Strain (in/in x 10^5)							
	1	2	3	5	6	7	8	9
i	0	0	0	0	0	0	0	0
ii	8	70	45	16	1	4	3	2
1	198	156	107	57	31	36	30	24
2	194	153	105	58	37	41	36	29
3	186	149	103	58	40	45	38	33
4	174	137	97	63	48	53	48	43
5	161	130	95	63	53	59	54	49
6	150	126	90	63	59	63	59	51
7	139	117	88	67	62	68	63	58
8	126	110	84	66	65	73	66	61
9	104	99	80	66	69	78	71	65
10	70	82	72	67	73	82	76	70
11	8	52	59	64	81	91	85	78
12	8	42	39	53	92	103	94	88
13	-37	5	10	36	99	111	104	96
14	-97	-43	-35	6	118	128	123	115
15	-212	-142	-120	-39	143	155	149	140
16								

(i) Before Release; (ii) After Release; (-) Tension

Inc.	Load (kips)	Deflection (in)		
		North	℄	South
1	0.12	0.00	0.00	0.00
2	.50	.03	.04	.03
3	1.00	.07	.08	.06
4	1.50	.13	.15	.12
5	2.00	.18	.22	.18
6	2.25	.23	.27	.22
7	2.50	.27	.32	.26
8	2.75	.30	.36	.29
9	3.00	.33	.41	.32
10	3.25	.39	.47	.37
11	3.50	.46	.59	.45
12	3.75	.58	.75	.56
13	4.00	.69	.90	.68
14	4.50	.93	1.22	.91
15	5.00	1.30	1.69	1.28
16	5.50		2.10	

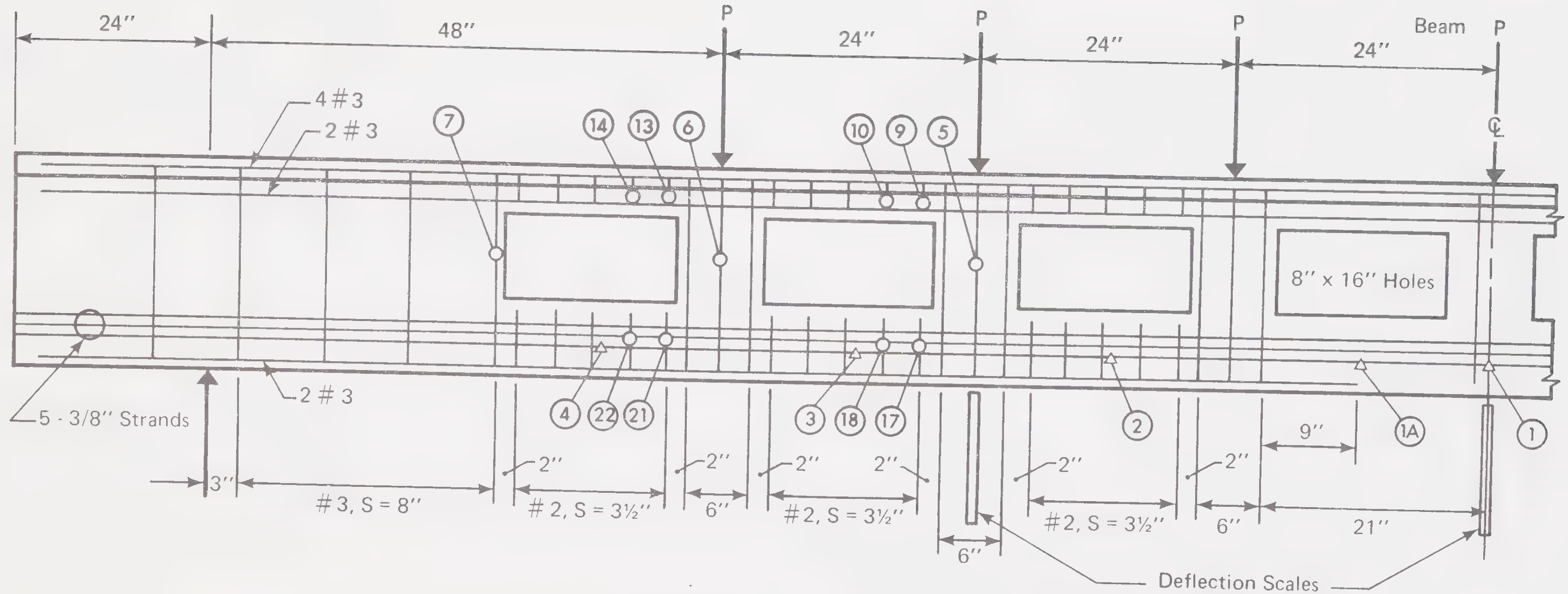


FIGURE B.9. Reinforcement Details and Strain Gage Locations for Beam 9–16–7L

TABLE B.10.1 ELECTRICAL STRAIN GAGE MEASUREMENTS (MICRO INCHES PER INCH)

BEAM NO. 10-16-6

Inc. No.	Load per Jack	Strain Gage Numbers								
		1	2	3	4	5	7	9	10	13
1	.148	0	0	0	0	0	0	0	0	0
2	2	50	50	35	25	-10	10	20	0	10
3	4	115	115	80	55	-25	5	60	- 5	50
4	6	185	190	135	85	-40	55	95	-10	125
5	7	220	225	160	95	-40	80	90	- 5	150
6	8	250	260	195	110	-40	100	80	- 5	160
7	9	310	320	255	120	-40	225	75	- 5	205
8	10	410	390	375	130	-30	300	75	0	260
9	11	1040	1350	460	140	-25	410	70	10	410
10	12	1400	1580	535	150	-30	580	70	15	450
11	13	1810	1790	615	160	-30	950	85	30	540
12	13.5	2095	2065	690	170	-30	1040	90	35	585
13	14	2410	2400	785	190	-35	1135	100	40	650
15	15	2970	3150	910	225	-35	1260	130	65	735
17	16	4240	4375	1625	400	-30	1420	185	140	860
19	17	6020	6920	2230	600	-40	1660	240	435	955
21	18	7115	9275	2470	635	-80	1850	250	565	1030
23	19	10000	13300	2840	870	-50	2135	290	690	1190
25	20	18250		3300	970	-35	4190	335	830	1340
30	20.5									

Inc. No.	Load per Jack	Strain Gage Numbers								
		14	17	18	21	22	42			
1	.148	0	0	0	0	0	0			
2	2	0	-15	-10	-15	-10	45			
3	4	0	-45	-10	-40	-20	180			
4	6	0	-75	- 5	-70	-35	400			
5	7	0	-80	- 5	-80	-40	530			
6	8	0	-85	0	-90	-45	650			
7	9	5	-95	15	-90	-60	850			
8	10	15	-95	50	-80	-50	1010			
9	11	75	-70	105	-75	-25	1255			
10	12	280	-15	165	-70	-15	1450			
11	13	395	50	290	-85	-10	1470			
12	13.5	430	85	325	-85	0	1420			
13	14	540	115	345	-80	10	1610			
15	15	825	155	390	-60	80	1900			
17	16	1230	270	480	-40	220	1870			
19	17		320	495	-10	330	2620			
21	18		300	505	- 5	355	2230			
23	19		300	535	0	405	2070			
25	20		280	540	0	440	1870			
30	20.5									

TABLE B.10.2 CENTERLINE STRAIN DISTRIBUTION AND BEAM DEFLECTIONS

Inc.	Strain (in/in x 10^5)							
	1	2	3	5	6	7	8	9
i	0	0	0	0	0	0	0	0
ii	63	59	36	11	0	3	4	1
1	160	137	100	49	30	33	31	27
2	151	133	98	52	35	39	35	32
3	138	121	93	55	41	45	42	39
4	124	114	90	55	48	51	49	48
5	122	108	88	56	52	55	52	51
6	113	103	84	55	58	59	57	56
7	96	94	76	60	61	63	61	60
8	72	79	73	60	64	67	65	65
9	-13	19	45	55	77	79	77	75
10	-76	-23	19	49	86	86	86	85
11	-134	-66	- 7	46	96	96	95	95
12	-181	-102	-31	36	102	101	101	102
13	-226	-137	-54	29	110	109	108	109
15	-314	-204	-99	25	121	118	118	120
17	-471	-343	-213	1	140	139	139	140
19	-707	-537	-377	-28	163	161	162	163
21	-895	-693	-570	-55	180	178	179	180
23	-1277	-951	-706	-105	207	206	208	209
25	-1575	-1281	-960	-172	252	240	241	242
30								

(i) Before Release; (ii) After Release; (-) Tension

Inc.	Load (kips)	Deflection (in)		
		North	℄	South
1	0.15	0.00	0.00	0.00
2	2	.04	.05	.05
3	4	.10	.12	.11
4	6	.17	.20	.17
5	7	.21	.24	.20
6	8	.24	.28	.24
7	9	.29	.34	.28
8	10	.34	.41	.34
9	11	.51	.62	.52
10	12	.68	.85	.69
11	13	.86	1.09	.86
12	13.5	.99	1.23	.98
13	14	1.11	1.41	1.11
15	15	1.37	1.74	1.37
17	16	1.79	2.32	1.81
19	17	2.40	3.04	2.33
21	18	2.90	3.71	2.91
23	19	3.64	4.91	3.67
25	20	4.48	5.71	4.49
30	20.5		9.36	

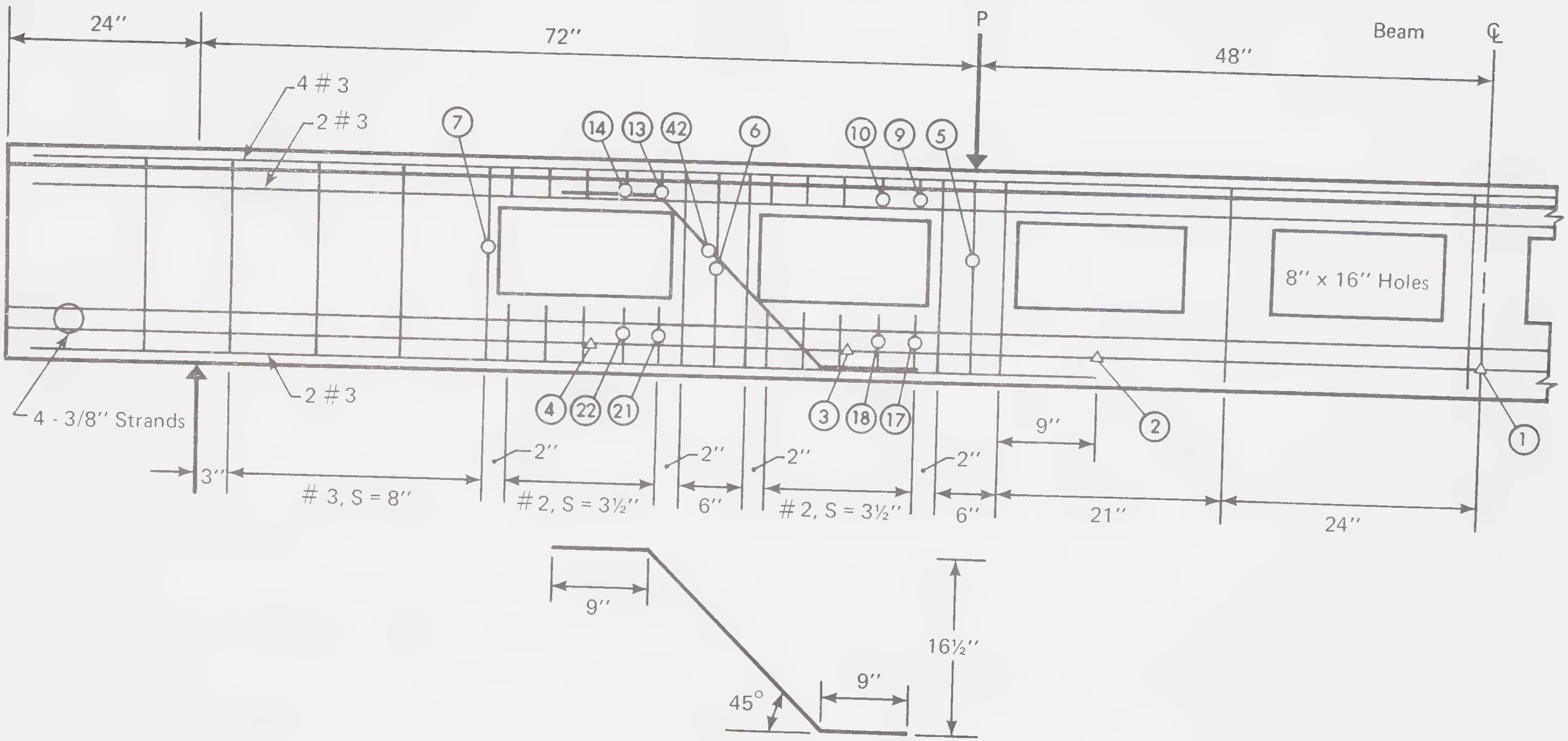


FIGURE B.10. Reinforcement Details and Strain Gage Locations for Beam 10–16–6

TABLE B.11.1 ELECTRICAL STRAIN GAGE MEASUREMENTS (MICRO INCHES PER INCH)

BEAM NO. 11-16-4-P

Inc. No.	Load per Jack	Strain Gage Numbers								
		1	2	4	6	7	13	14	14A	21
1	.15	0	0	0	0	0	0	0	0	0
2	2	40	40	40	10	35	10	5	0	5
3	4	90	90	90	20	90	25	10	0	20
4	6	140	140	145	30	165	40	20	- 5	30
5	8	185	185	200	50	350	50	30	0	40
6	9	220	220	235	60	440	60	35	5	50
7	10	245	245	270	70	490	70	40	5	55
8	11	275	275	320	90	550	80	45	5	65
9	12	305	305	365	115	595	85	50	10	70
10	13	335	335	415	130	650	95	60	10	90
11	14	370	370	525	165	750	110	70	20	110
12	15	415	410	610	180	820	135	85	25	125
13	16	485	480	785	225	910	190	110	35	165
14	18	865	650	980	270	1030	335	175	60	210
15	20	1520	1150	1255	320	1225	640	380	80	305
16	22	2120	1630	1580	345	1545	1160	1065	405	375
18	24	2770	2365	1910	370	1795	1380	1300	560	455
20	26	3630	3355	2270	420	1945	1690	1580	700	515
22	28	4720	4550	3030	510	2060	2020	1875	895	685
23	29						2000			

Inc. No.	Load per Jack	Strain Gage Numbers								
		22	22A							
1	.15	0	0							
2	2	10	20							
3	4	20	50							
4	6	35	80							
5	8	45	110							
6	9	55	130							
7	10	60	140							
8	11	70	170							
9	12	75	195							
10	13	95	230							
11	14	130	275							
12	15	155	340							
13	16	185	440							
14	18	265	625							
15	20	370	750							
16	22	520	910							
18	24	625	1055							
20	26	690	1185							
22	28	750	1470							
23	29									

TABLE B.11.2 CENTERLINE STRAIN DISTRIBUTION AND BEAM DEFLECTIONS

Inc.	Strain (in/in x 10^5)							
	1	2	3	5	6	7	8	9
i	0	0	0	0	0	0	0	0
ii	77	64	51	13	1	3	4	1
1	187	160	130	52	32	40	41	34
2	186	159	130	45	36	44	45	38
3	179	157	130	48	40	49	49	42
4	171	151	126	47	44	53	54	46
5	164	145	122	50	48	57	56	51
6	160	142	121	52	51	60	60	54
7	159	141	121	52	53	63	63	56
8	153	135	115	52	56	66	65	59
9	149	134	114	53	57	67	67	61
10	145	131	112	53	60	70	71	63
11	136	124	107	55	63	73	73	65
12	133	122	105	54	64	75	76	69
13	122	114	100	55	68	79	80	73
14	75	85	93	56	75	85	86	79
15	-39	4	32	50	87	96	97	91
16	-145	-67	-8	47	97	104	105	101
18	-222	-127	-46	44	109	116	117	114
20	-320	-208	-108	41	122	130	133	129
22	-447	-316	-296	29	136	145	147	144
23								

(i) Before Release; (ii) After Release; (-) Tension

Inc.	Load (kips)	Deflection (in)		
		North	℄	South
1	0.15	0.00	0.00	0.00
2	2	.03	.04	.03
3	4	.08	.08	.07
4	6	.11	.13	.11
5	8	.15	.17	.15
6	9	.18	.20	.17
7	10	.20	.22	.19
8	11	.23	.25	.22
9	12	.25	.28	.24
10	13	.27	.31	.27
11	14	.30	.35	.30
12	15	.33	.38	.33
13	16	.37	.42	.37
14	18	.45	.53	.45
15	20	.67	.81	.66
16	22	.92	1.12	.93
18	24	1.18	1.45	1.19
20	26	1.48	1.84	1.51
22	28	1.87	2.32	1.92
23	29			

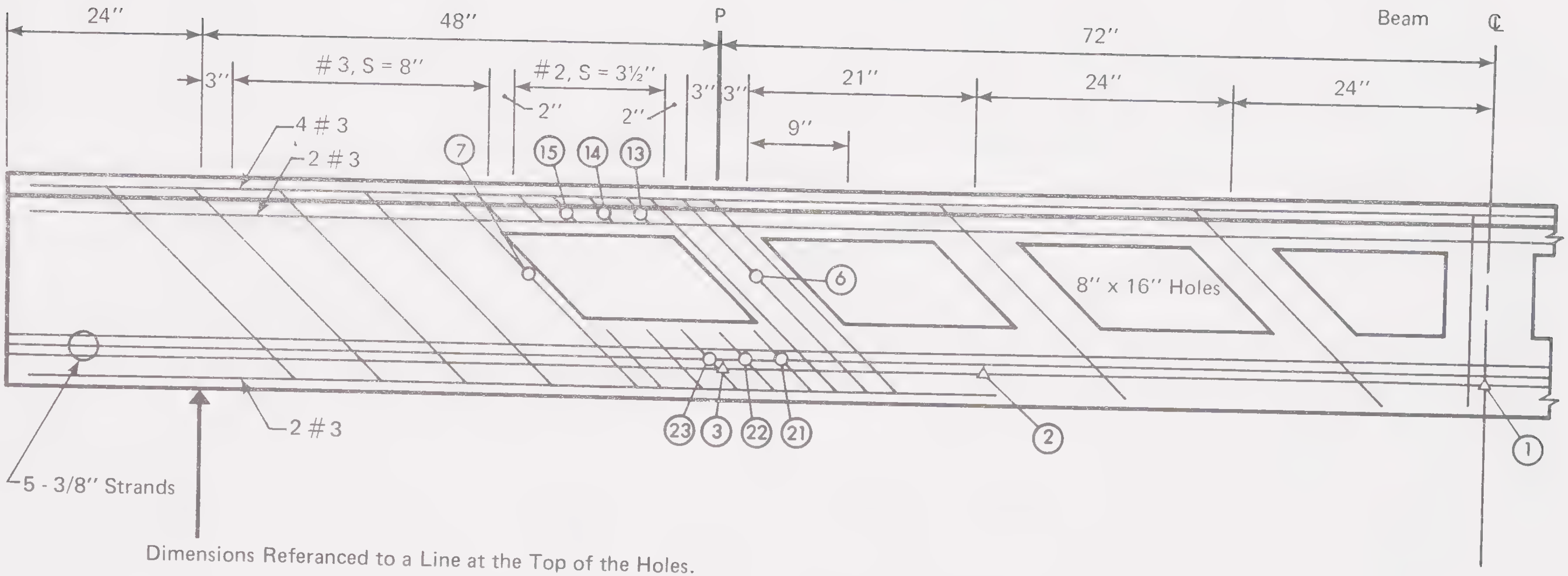


FIGURE B.11. Reinforcement Details and Strain Gage Locations for Beam 11–16–4–P

TABLE B.12.1 ELECTRICAL STRAIN GAGE MEASUREMENTS (MICRO INCHES PER INCH)

BEAM NO. 12-16-6-P

Inc. No.	Load per Jack	Strain Gage Numbers								
		1	2	4	5	6	7	9	10	11
1	0.15	0	0	0	0	0	0	0	0	0
2	2	60	60	35	10	30	50	10	0	- 5
3	4	145	135	85	20	105	170	10	0	-15
4	6	230	220	135	35	230	345	15	0	-25
5	8	315	305	190	60	460	535	25	10	-30
6	9	370	360	225	80	575	600	35	10	-40
7	10	435	420	255	95	670	685	40	20	-40
8	11	585	560	320	120	780	770	60	25	-40
9	12	850	740	370	135	860	850	70	35	-40
10	13	1335	1130	460	140	940	950	90	45	-35
11	14	1870	1460	625	150	1050	1050	110	60	-30
12	15	2200	1910	745	160	1130	1120	130	65	-30
13	16	2750	2470	885	185	1220	1210	185	80	-25
14	18	3825	2540	1140	240	1365	1350	280	95	-20
15	20	5805	9200	1455	345	1600	1570	620	160	5
16	22			1730	475	2060	1975		1090	530
17	23			1840	540		1920		1215	720
18	23			1860	570		1900		1395	870
19	24			1935	610		1900		1550	935
20	25									

Inc. No.	Load per Jack	Strain Gage Numbers								
		13	14	15	17	18	19	21	22	23
1	0.15	0	0	0	0	0	0	0	0	0
2	2	5	0	- 5	20	20	30	10	0	15
3	4	15	0	-10	40	50	60	20	0	40
4	6	20	- 5	-25	60	80	100	30	-10	60
5	8	25	-10	-30	80	115	120	60	-20	80
6	9	30	-10	-35	105	160	200	80	-25	100
7	10	35	-10	-40	125	200	275	100	-30	110
8	11	50	-10	-40	160	220	435	110	-35	140
9	12	70	-10	-40	230	340	585	185	-40	165
10	13	90	-10	-40	320	420	740	205	-30	205
11	14	125	- 5	-40	430	565	910	225	-20	305
12	15	155	0	-40	525	665	1050	245	0	440
13	16	210	5	-30	620	790	1210	280	135	600
14	18	300	25	-20	760	960	1420	325	255	865
15	20	400	50	- 5	1040	1290	1620	385	360	1110
16	22	465	90	5	720	1430	1820	365	420	1280
17	23	530	140	35	600	1460	1810	340	420	1330
18	23	590	250	80	570	1425	1770	335	420	1310
19	24	605	285	95	560	1550	1875	370	435	1360
20							1920			1400

TABLE B.12.2 CENTERLINE STRAIN DISTRIBUTION AND BEAM DEFLECTIONS

Inc.	Strain (in/in x 10^5)							
	1	2	3	5	6	7	8	9
i	0	0	0	0	0	0	0	0
ii	77	66	50	3	0	3	1	1
1	193	160	150	43	30	34	31	37
2	191	158	129	46	37	43	38	43
3	179	150	123	49	42	47	44	49
4	168	139	121	52	49	54	51	56
5	156	131	112	52	55	61	63	63
6	150	125	109	55	61	66	65	67
7	139	116	104	56	64	69	68	71
8	119	104	96	57	69	74	73	77
9	85	85	87	57	74	79	79	81
10	14	38	66	53	82	88	87	89
11	-93	-32	30	49	90	96	94	97
12	-152	-73	9	44	96	102	101	106
13	-221	-124	-19	40	108	112	111	116
14	-367	-228	-79	29	128	132	132	137
15	-629	-405	-178	16	152	157	157	163
16	-1136	-775	-421	-10	198	205	204	210
17	-1731	-1325	-899	-108	242	251	251	258
18	-1891	-1475	-1064	-141	256	265	265	271
19		-1885	-1609	-261	314	324	322	330
20			-1919					

(i) Before Release; (ii) After Release; (-) Tension

Inc.	Load (kips)	Deflection (in)		
		North	℄	South
1	0.15	0.00	0.00	0.00
2	2	.04	.05	.04
3	4	.10	.12	.09
4	6	.16	.18	.15
5	8	.22	.25	.21
6	9	.26	.32	.26
7	10	.30	.32	.29
8	11	.36	.41	.34
9	12	.41	.47	.40
10	13	.52	.61	.50
11	14	.67	.81	.66
12	15	.80	.97	.77
13	16	.97	1.18	.94
14	18	1.26	1.57	1.25
15	20	1.76	2.28	1.74
16	22	2.99	3.74	2.97
17	23	3.95	5.00	3.95
18	23	4.25	5.44	4.29
19	24	5.65	7.55	5.77
20	25		8.45	

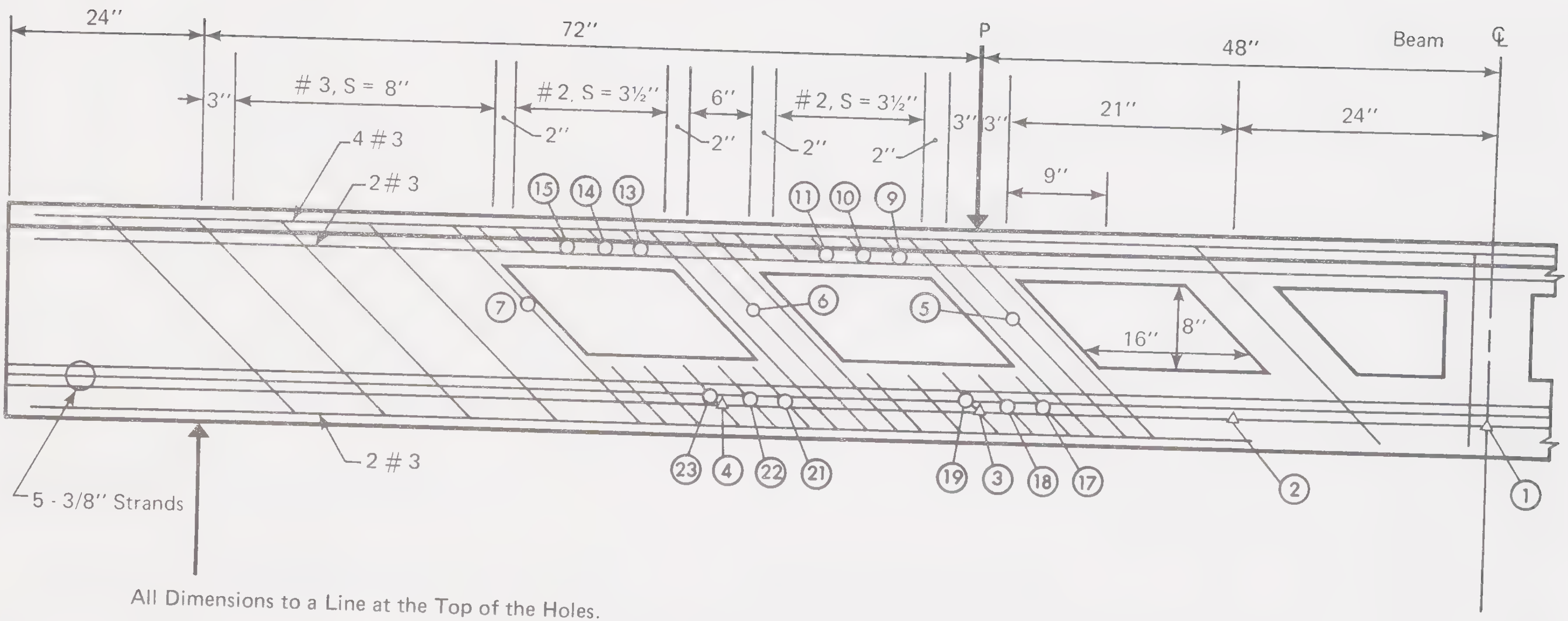


FIGURE B.12. Reinforcement Details and Strain Gage Locations for Beam 12–16–6–P

TABLE B.13.1 ELECTRICAL STRAIN GAGE MEASUREMENTS (MICRO INCHES PER INCH)

BEAM NO. 13-16-6-P

Inc. No.	Load per Jack	Strain Gage Numbers								
		1	2	3	4	5	6	7	9	10
1	0.15	0	0	0	0	0	0	0	0	0
2	2	60	55	55	35	5	40	30	5	0
3	4	140	135	130	80	15	235	80	20	5
4	6	225	220	215	135	35	845	215	60	15
5	8	315	300	325	190	65	1135	365	135	25
6	9	370	365	435	235	235	1240	500	180	30
7	10	440	435	575	270	400	1365	655	200	40
8	11	670	625	770	320	600	1580	855	210	45
9	12	970	820	935	370	730	1720	1005	230	55
10	13	1345	1080	1090	450	840	1820	1130	250	65
11	14	1820	1620	1315	635	1035	2005	1285	290	75
12	15	2135	1975	1475	735	1135	2140	1390	320	85
13	17	3150	3160	1940	1055	1375	8800	1690	405	115
14	19	4800	4230	2705	1470	1800	19480	2065	630	260
15	20	5585	5030	3050	1620	1920	19700	2110	910	520
16	21	7790		3475	1855	2035	19800	2120	1400	1225
17	22	8550		3670	1875	2105	21100	2095	1530	1390
18	23	10260		4070	1935	2200	23600	2130	1770	1700
19	24									

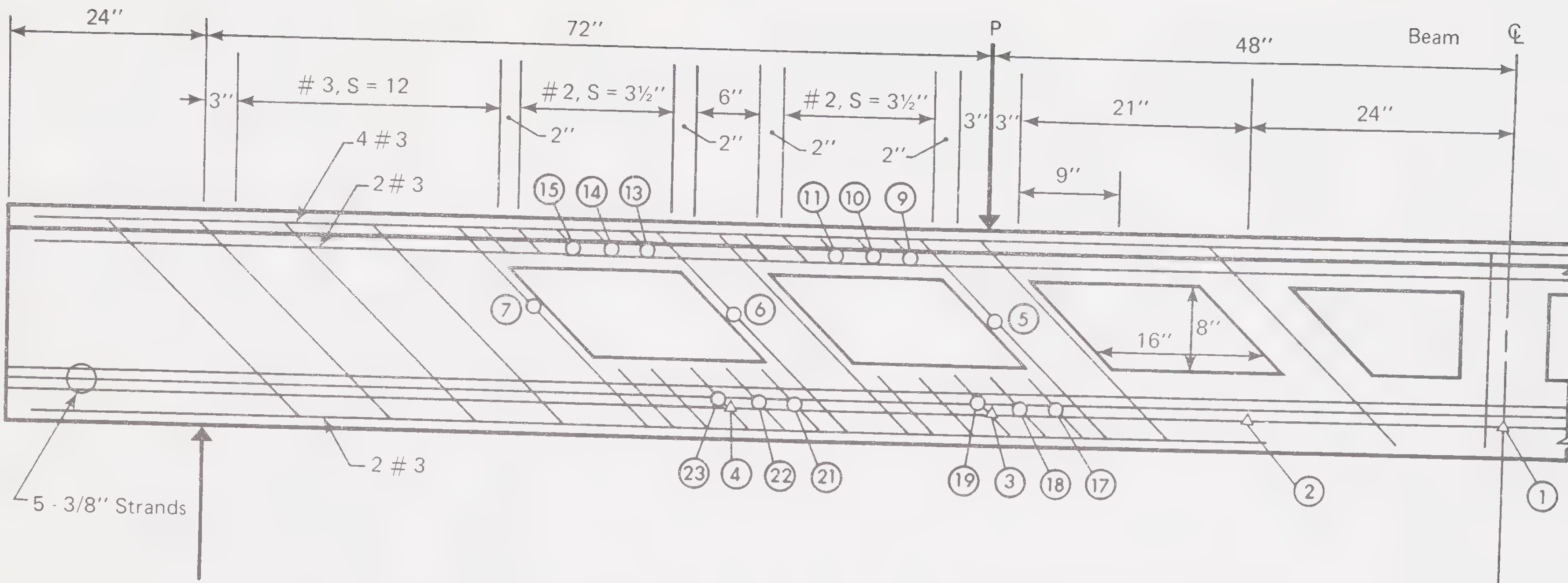
Inc. No.	Load per Jack	Strain Gage Numbers									
		11	13	14	15	17	18	19	21	22	23
1	0.15	0	0	0	0	0	0	0	0	0	0
2	2	-40	5	5	- 5	10	15	30	10	0	20
3	4	-255	10	5	-10	20	35	70	25	0	50
4	6	-370	10	0	-20	30	50	110	55	- 5	75
5	8	-340	15	0	-30	40	75	155	180	5	90
6	9	-230	20	- 5	-40	50	110	225	260	20	105
7	10	-120	35	0	-40	65	150	305	290	20	115
8	11	-270	55	0	-40	90	215	460	360	25	130
9	12	-305	75	0	-45	135	285	600	400	30	160
10	13	-320	100	0	-45	190	375	735	430	25	200
11	14	-320	150	5	-45	250	500	900	465	30	310
12	15	-325	190	10	-45	290	575	1020	490	40	390
13	17	-340	295	25	-35	370	735	1310	545	140	660
14	19	-15	330	30	-40	410	1010	1715	600	205	925
15	20	-110	350	40	-40	365	1105	1840	570	215	1005
16	21	600	425	60	-35	390	1065	1915	570	260	1110
17	22	-430	460	80	-25	415	1040	1975	685	290	1110
18	23	-240	485	95	-15	460	1035	2120	775	310	1145
19	24										

TABLE B.13.2 CENTERLINE STRAIN DISTRIBUTION AND BEAM DEFLECTIONS

Inc.	Strain (in/in x 10^5)							
	1	2	3	5	6	7	8	9
i	0	0	0	0	0	0	0	0
ii	88	72	55	11	- 3	0	1	2
1	217	176	130	68	29	41	37	34
2	212	173	129	72	35	47	43	39
3	201	165	125	73	42	53	50	45
4	189	158	118	76	49	60	56	51
5	180	150	112	79	57	68	63	59
6	170	143	109	83	62	72	68	65
7	159	136	106	83	67	76	72	69
8	132	121	98	83	73	82	78	75
9	98	102	92	81	80	87	84	81
10	50	71	78	78	87	93	90	87
11	-24	21	53	76	94	100	97	97
12	-57	- 4	40	74	102	107	104	105
13	-166	-103	-13	61	122	128	124	127
14	-289	-240	-89	36	147	151	150	153
15	-365	-324	-136	26	159	165	163	166
16	-542	-537	-256	- 2	183	181	177	191
17	-622	-630	-308	-14	195	202	198	203
18	-742	-782	-399	-37	214	222	218	224
19								

(i) Before Release; (ii) After Release; (-) Tension

Inc.	Load (kips)	Deflection (in)		
		North	℄	South
1	0.15	0.00	0.00	0.00
2	2	.04	.04	.04
3	4	.10	.11	.10
4	6	.16	.18	.16
5	8	.22	.25	.23
6	9	.27	.30	.27
7	10	.31	.36	.31
8	11	.37	.43	.37
9	12	.44	.50	.44
10	13	.55	.65	.54
11	14	.72	.86	.71
12	15	.82	.99	.81
13	17	1.14	1.39	1.12
14	19	1.58	1.90	1.61
15	20	1.85	2.28	1.87
16	21	2.48	2.86	1.48
17	22	2.74	3.41	1.69
18	23	3.15	3.84	3.16
19	24		6.48	



All Dimensions to a Line at the Top of the Holes.

FIGURE B.13. Reinforcement Details and Strain Gage Locations for Beam 13–16–6–P

TABLE B.14.1 ELECTRICAL STRAIN GAGE MEASUREMENTS (MICRO INCHES PER INCH)

BEAM NO. 14-12-6

Inc. No.	Load per Jack	Strain Gage Numbers								
		1	2	3	4	5	6	7		
1	0.15	0	0	0	0	0	0	0		
2	2	55	55	50	30	10	30	30		
3	4	135	140	130	80	25	75	75		
4	6	225	230	215	125	40	125	130		
5	8	300	320	295	170	55	180	250		
6	9	360	400	360	200	65	340	350		
7	10	410	460	415	230	75	395	405		
8	11	525	590	500	265	90	500	470		
9	12	650	700	590	300	90	585	540		
10	13	975	900	800	340	95	655	595		
11	14	1610	1270	1055	400	110	750	680		
12	15	2000	1545	1240	455	130	825	750		
13	16	2560	1890	1500	550	160	910	875		
15	18	3555	2435	1900	775	210	1050	1045		
17	20	5560	5350	2470	1120	330	1205	1270		
19	22	10800		3320	1450	430	1375	1545		
20	23	11900		3410	1450	440	1410	1760		
21	24	12400		3610	1480	440	1475	1875		
22	25			3770	1460	445	1480	1925		
23	25			4050	1600	455	1630	2005		

TABLE B.14.2 CENTERLINE STRAIN DISTRIBUTION AND BEAM DEFLECTIONS

Inc.	Strain (in/in x 10^5)							
	1	2	3	4	5	6	7	8
i	0	0	0	0	0	0	0	0
ii	84	69	45	6	5	- 2	1	2
1	201	166	117	42	54	32	42	45
2	200	166	118	42	57	38	48	51
3	188	158	113	41	59	44	53	57
4	176	148	108	42	64	51	61	64
5	165	140	103	43	66	58	67	69
6	158	135	99	43	69	63	71	74
7	150	129	96	42	69	67	75	76
8	135	120	91	42	69	72	79	83
9	113	108	85	42	70	76	83	87
10	66	79	72	42	70	82	88	92
11	- 4	26	31	38	69	91	97	100
12	-47	-12	16	32	66	100	104	108
13	-109	-68	-32	33	59	108	113	118
15	-215	-175	-126	-23	50	127	131	137
17	-420	-374	-291	-99	27	156	161	167
19	-950	-825	-742	-327	-58	221	223	236
20	-1045	-1023	-818	-365	-72	236	243	253
21	-1280	-1305	-1027	-465	-111	265	273	283
22	-1660	-2045	-1617	-665	-191	311	315	328
23								

(i) Before Release; (ii) After Release; (-) Tension

Inc.	Load (kips)	Deflection (in)		
		North	℄	South
1	0.15	0.00	0.00	0.00
2	2	.03	.04	.03
3	4	.08	.10	.08
4	6	.14	.17	.14
5	8	.20	.24	.20
6	9	.23	.29	.24
7	10	.27	.32	.27
8	11	.32	.38	.32
9	12	.37	.45	.37
10	13	.45	.56	.46
11	14	.63	.79	.64
12	15	.76	.95	.76
13	16	.93	1.16	.93
15	18	1.23	1.53	1.21
17	20	1.80	2.26	1.77
19	22	3.33	4.23	3.31
20	23	3.60	4.59	3.59
21	24	4.22	5.36	4.18
22	25	5.49	7.32	5.30
23	25		10.28	

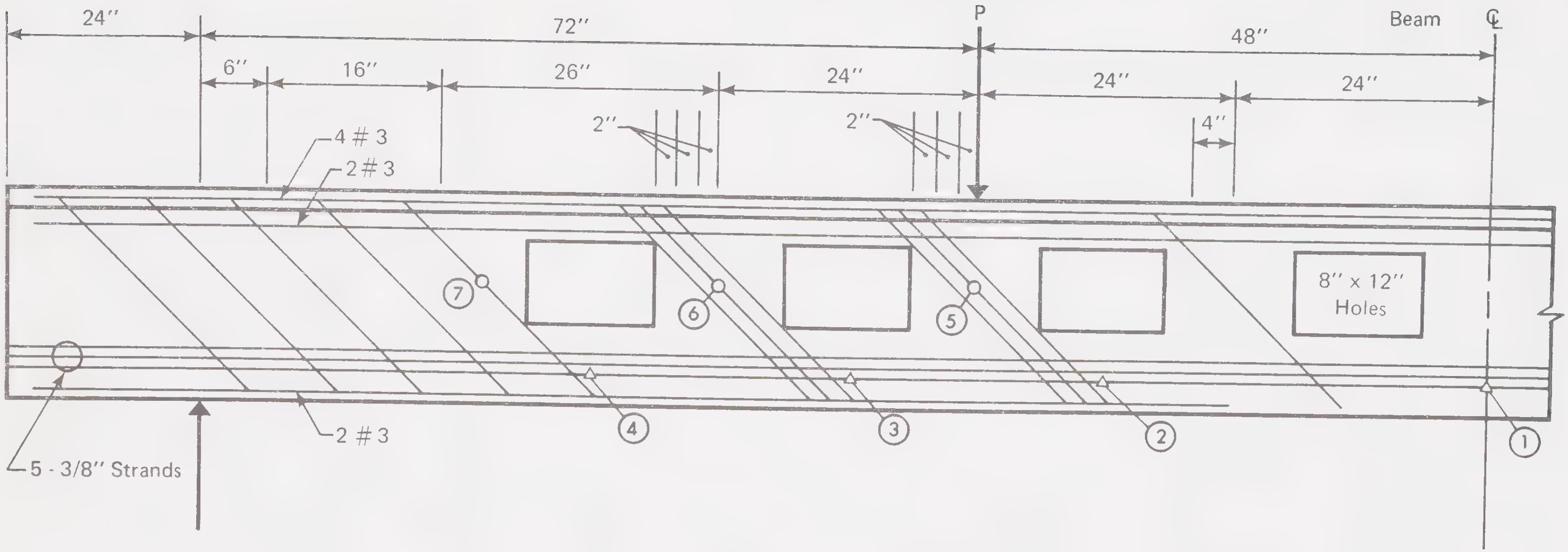


FIGURE B.14. Reinforcement Details and Strain Gage Locations for Beam 14–12–6

TABLE B.15.1 ELECTRICAL STRAIN GAGE MEASUREMENTS (MICRO INCHES PER INCH)

BEAM NO. 15-12-6

Inc. No.	Load per Jack	Strain Gage Numbers								
		1	2	3	4	5	6	7		
1	0.15	0	0	0	0	0	0	0		
2	2	65	65	0	25	- 5	0	10		
3	4	150	150	60	60	-10	10	15		
4	6	240	235	120	95	-15	25	20		
5	8	330	320	185	130	-10	70	40		
6	9	395	380	220	155	-10	85	90		
7	10	450	430	255	170	-10	100	145		
8	11	565	515	295	200	- 5	120	250		
9	12	650	580	345	220	0	160	350		
10	13	760	760	550	245	15	1090	505		
11	14	1660	1205	900	285	30	1260	645		
12	15	2030	1510	1130	315	30	1365	700		
13	16	2685	2710	2430	360	25	1480	840		
14	17	3200	3160	3560	395	20	1520	890		
15	18	3260	3240	4340	440	15	1570	975		
16	19						1700			

TABLE B.15.2 CENTERLINE STRAIN DISTRIBUTION AND BEAM DEFLECTIONS

Inc.	Strain (in/in x 10^5)							
	1	2	3	4	5	6	7	8
i	0	0	0	0	0	0	0	0
ii								
1	179	143	107	31	44	26	40	32
2	175	140	105	35	46	32	42	36
3	165	131	100	33	48	36	48	42
4	153	123	94	33	52	42	55	50
5	142	115	89	34	52	49	61	56
6	133	110	87	34	54	54	65	60
7	126	105	85	34	56	57	68	64
8	107	95	80	36	56	62	74	69
9	84	83	85	34	56	66	77	72
10	43	58	65	34	58	69	79	75
11	-142	-49	16	34	55	79	87	83
12	-219	-90	-11	29	52	87	95	91
13	-353	-166	-54	21	58	99	105	101
14	-469	-237	-91	4	44	108	114	111
15	-577	-306	-131	-16	40	117	124	120
16								

(i) Before Release; (ii) After Release; (-) Tension

Inc.	Load (kips)	Deflection (in)		
		North	℄	South
1	0.15	0.00	0.00	0.00
2	2	.04	.05	.04
3	4	.09	.11	.09
4	6	.15	.17	.14
5	8	.20	.25	.20
6	9	.25	.29	.24
7	10	.27	.32	.27
8	11	.33	.39	.33
9	12	.37	.45	.39
10	13	.46	.54	.45
11	14	.66	.77	.66
12	15	.78	.96	.79
13	16	1.01	1.23	1.01
14	17	1.18	1.45	1.17
15	18	1.37	1.70	1.36
16	19		2.16	

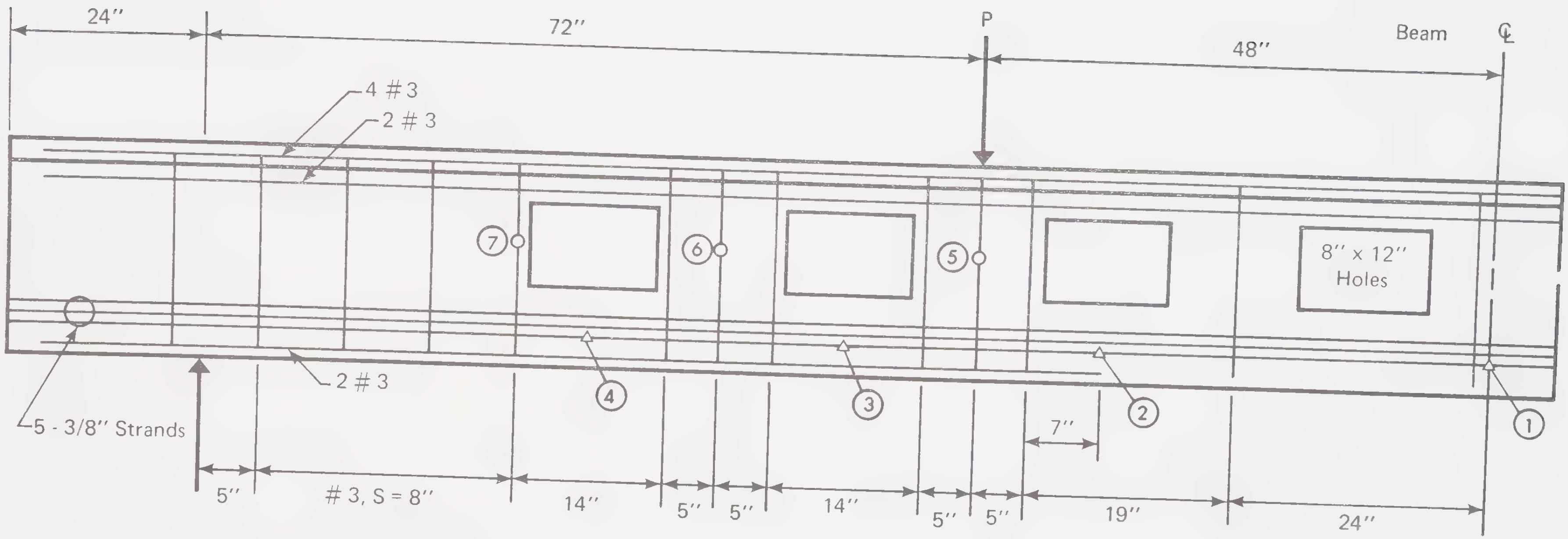


FIGURE B.15. Reinforcement Details and Strain Gage Locations for Beam 15–12–6

TABLE B.16.1 ELECTRICAL STRAIN GAGE MEASUREMENTS (MICRO INCHES PER INCH)

BEAM NO. 16-12-6

Inc. No.	Load per Jack	Strain Gage Numbers								
		1	3	4	5	6	7	9	10	13
1	0.15	0	0	0	0	0	0	0	0	0
2	2	55	40	20	0	0	5	15	0	10
3	4	140	105	55	-10	10	25	45	5	20
4	6	220	175	90	-10	40	60	80	10	20
5	8	300	245	125	- 5	70	110	85	15	25
6	9	340	280	140	- 5	75	105	75	15	25
7	10	410	345	170	0	90	105	70	15	30
8	11	560	455	195	10	140 - 575	120	60	20	35
9	12	810	610	225	20	645	180	55	20	50
10	13	1070	800	260	30	710	300	65	25	60
11	14	1690	1090	310	50	760	925	95	30	75
12	15	2055	1280	355	45	795	1075	140	40	90
13	16	2620	1555	450	45	825	1210	240	60	115
14	17	3085	1735	545	45	850	1300	285	100	130
15	18	3590	1950	770	45	875	1390	350	155	175
17	20	5340	2735	970	40	870	1745	915	905	255
18	21	7300	3410	1075	40	895	2005	1125	1460	265
19	22	12290	3920	1180	35	910	2270	1280	2320	305
20	23	14000	4150	1155	25	930	3680	1365	7300	715
21	24	14500	4435	1165	30	1680		1400	14500+	850
22	24							1360		

Inc. No.	Load per Jack	Strain Gage Numbers			
		14	17	18	22
1	0.15	0	0	0	0
2	2	0	- 5	-10	0
3	4	0	-15	-20	- 5
4	6	0	-25	-30	-10
5	8	0	-35	-40	-15
6	9	0	-40	-45	-15
7	10	0	-60	-35	-20
8	11	0	-70	40	-30
9	12	5	-70	110	-30
10	13	5	-25	150	0
11	14	5	25	235	30
12	15	5	70	305	60
13	16	10	125	380	120
14	17	15	155	420	165
15	18	25	180	470	485
17	20	50	335	570	760
18	21	55	415	650	905
19	22	80	475	780	985
20	23	420	520	850	1060
21	24	490	520	900	1110
22	24				

TABLE B.16.2 CENTERLINE STRAIN DISTRIBUTION AND BEAM DEFLECTIONS

Inc.	Strain (in/in x 10^5)							
	1	2	3	4	5	6	7	8
i	0	0	0	0	0	0	0	0
ii	86	65	45	6	5	- 4	- 1	2
1	199	158	116	39	52	28	40	39
2	196	155	113	38	51	33	42	43
3	186	147	110	38	54	40	49	49
4	173	138	103	36	55	47	56	56
5	163	131	98	37	59	53	61	62
6	156	125	94	37	59	56	65	65
7	146	119	92	37	60	62	69	69
8	123	106	85	37	62	68	74	74
9	94	91	77	37	64	71	78	79
10	67	72	67	37	62	81	85	86
11	-40	13	36	34	59	87	91	92
12	-108	-35	10	30	59	94	97	99
13	-182	-111	-22	16	54	105	108	109
14	-238	-167	-53	1	51	113	115	116
15	-285	-225	-90	-25	43	115	126	128
17	-453	-462	-208	-97	18	154	156	158
18	-620	-679	-324	-171	- 9	178	181	185
19								
20								
21								

(i) Before Release; (ii) After Release; (-) Tension

Inc.	Load (kips)	Deflection (in)		
		North	℄	South
1	0.15	0.00	0.00	0.00
2	2	.04	.05	.04
3	4	.09	.12	.10
4	6	.15	.19	.16
5	8	.22	.25	.20
6	9	.24	.28	.24
7	10	.28	.34	.30
8	11	.34	.40	.35
9	12	.41	.48	.40
10	13	.51	.59	.51
11	14	.72	.85	.73
12	15	.86	1.03	.87
13	16	1.09	1.29	1.09
14	17	1.23	1.51	1.23
15	18	1.41	1.75	1.41
17	20	2.05	2.54	2.07
18	21	2.68	3.30	2.64
19	22	3.70	4.52	3.60
20	23	4.21	5.44	4.23
21	24	4.89	6.17	4.78

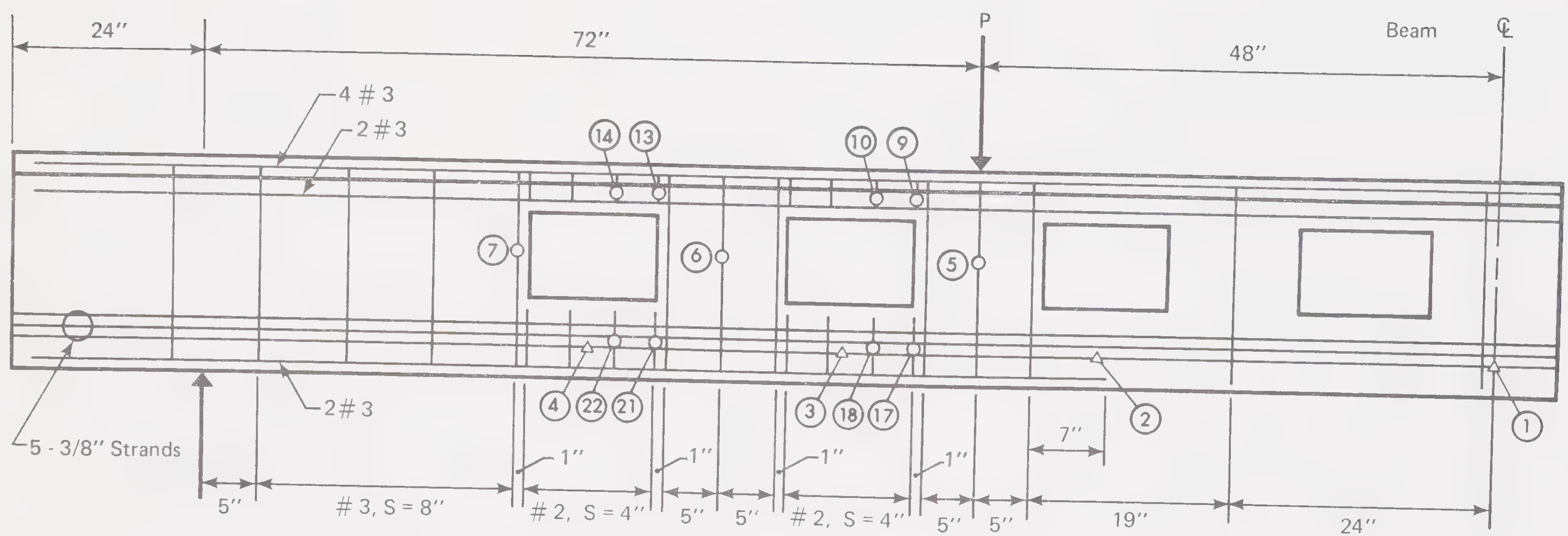


FIGURE B.16. Reinforcement Details and Strain Gage Locations for Beam 16–12–6

TABLE B.17.1 ELECTRICAL STRAIN GAGE MEASUREMENTS (MICRO INCHES PER INCH)

BEAM NO. 17-16-4

Inc. No.	Load per Jack	Strain Gage Number								
		1	4	6	7	8	13	15	16	21
1	0.15	0	0	0	0	0	0	0	0	0
2	2	30	20	- 5	10	0	10	0	- 5	-10
3	4	80	45	-15	40	10	30	5	-15	-30
4	6	125	70	-30	80	20	90	5	-25	-45
5	8	170	100	-35	85	35	125	10	-35	-60
6	10	220	135	-40	105	45	160	10	-50	-85
7	12	275	170	-40	155	55	150	15	-60	-110
8	14	340	220	-40	200	55	130	20	-65	-155
10	16	415	270	-20	485	900	150	25	-75	-150
12	18	710	320	240	645	1070	160	30	-70	-120
14	20	1380	470	305	800	1260	255	55	-70	-105
15	21	1680	670	320	880	1350	325	85	-60	-100
16	22	1890	750	310	935	1415	355	100	-60	-100
18	24	2450	1025	325	1155	1545	460	220	-40	-90
20	26	3205	1290	365	1385	1680	555	400	-30	80
21	27	3610	1470	370	1445	1750	555	495	-20	180
22	28	4000	1670	370	1485	1820	540	655	-15	180
23	29	4865	2160	360	1505	1950	480	885	0	230
24	30	6100	2710	340	1475	2105	425	1045	70	210
25	31	7800		0	1620	2275	420	1215	210	200

Inc. No.	Load per Jack	Strain Gage Number									
		23	24	29	30	31	32	37	38	39	40
1	0.15	0	0	0	0	0	0	0	0	0	0
2	2	- 5	0	-20	10	-15	-35	-10	45	40	10
3	4	-10	10	-50	40	-30	-85	-30	120	100	25
4	6	-15	15	-75	80	-55	-150	-50	200	175	40
5	8	-20	10	-100	120	-65	-205	-70	280	235	60
6	10	-25	5	-130	200	-180	-290	-110	405	305	85
7	12	-30	5	-160	325	-90	-380	-150	600	400	110
8	14	-35	40	-200	490	-80	-490	-225	870	525	155
10	16	-35	125	-240	750	-70	-635	-275	1360	700	180
12	18	-35	430	-265	850	-40	-780	-300	1735	1060	200
14	20	30	595	-295	1135	30	-975	-240	2090	1425	280
15	21	440	625	-305	1340	70	-1650	-205	2310	1635	350
16	22	570	650	-320	1420	95	-2165	-185	2475	1790	460
18	24	755	710	-310	1690	150	-3475	-125	2895	2220	630
20	26	925	740	-305	2050	230	-4850	-35	3210	2680	750
21	27	980	755	-245	2185	290	-5800	- 5	3250	2710	805
22	28	1025	765	-125	2260	350	-7100	15	3240	2740	865
23	29	1075	745	-30	2285	415	-10100	40	4500*	2800	945
24	30	1115	700	-25	2275	570	-18800	-70	13500	2800*	1200*
25	31	1160	840	-45	2290	865	-21400	-350	23300	3015	1370

TABLE B.17.2 CENTERLINE STRAIN DISTRIBUTION AND BEAM DEFLECTIONS

Inc.	Strain (in/in x 10^5)							
	1	2	3	5	6	7	8	9
i	0	0	0	0	0	0	0	0
ii	91	74	51	12	- 3	- 2	- 3	- 3
1	169	148	110	45	26	27	26	26
2	170	149	110	45	28	30	28	29
3	162	143	107	47	32	34	33	35
4	155	137	103	48	36	39	38	42
5	150	132	101	51	41	43	42	47
6	142	126	97	51	44	47	45	52
7	133	121	95	54	49	52	51	58
8	124	115	91	54	55	57	56	63
10	110	105	85	56	60	63	62	69
12	75	86	77	57	65	68	67	74
14	- 1	45	61	58	71	74	73	81
15	-77	12	44	54	79	82	80	89
16	-138	-11	31	52	85	87	86	95
18	-223	-56	- 5	40	101	103	103	111
20	-323	-109	-44	21	118	122	121	129
21	-380	-139	-66	11	125	128	129	136
22	-438	-167	-87	4	134	137	135	144
23	-486	-213	-120	-11	145	149	148	156
24	-750	-316	-185	-32	158	162	162	169
25								

(i) Before Release; (ii) After Release; (-) Tension

Inc.	Load (kips)	Deflection (in)		
		North	℄	South
1	0.15	0.00	0.00	0.00
2	2	.02	.00	.00
3	4	.04	.03	.03
4	6	.07	.06	.05
5	8	.09	.09	.07
6	10	.11	.12	.10
7	12	.15	.16	.13
8	14	.18	.20	.16
10	16	.23	.26	.21
12	18	.28	.32	.26
14	20	.37	.44	.36
15	21	.47	.58	.46
16	22	.54	.67	.50
18	24	.70	.90	.68
20	26	.90	1.17	.88
21	27	1.01	1.31	1.00
22	28	1.13	1.48	1.12
23	29	1.33	1.73	1.33
24	30	1.63	2.09	1.60
25	31		2.75	

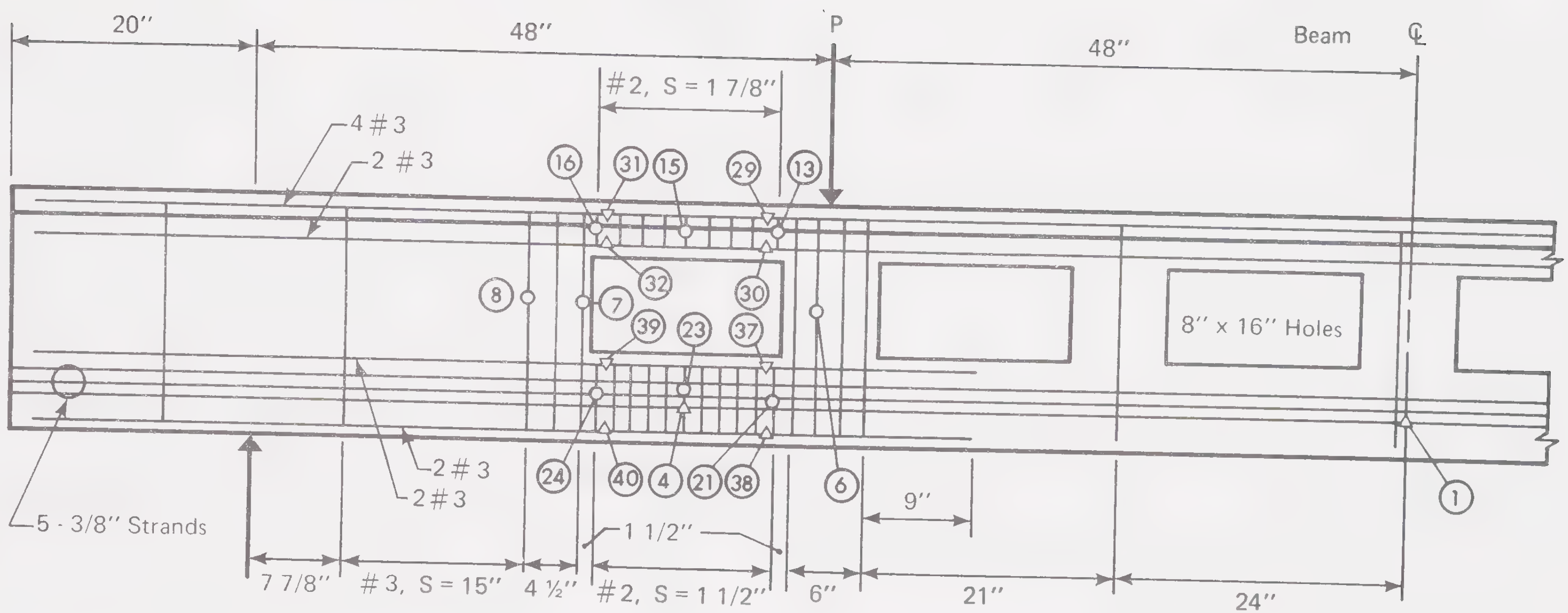


FIGURE B.17. Reinforcement Details and Strain Gage Locations for Beam 17–16–4

TABLE B.18.1 ELECTRICAL STRAIN GAGE MEASUREMENTS (MICRO INCHES PER INCH)

BEAM NO. 18-16-4

Inc. No.	Load per Jack	Strain Gage Number								
		1	4	6	7	8	13	15	16	21
1	0.17	0	0	0	0	0	0	0	0	0
2	2.5	55	40	-15	10	5	35	5	-10	-25
3	5	110	75	-30	15	15	60	10	-15	-50
4	7.5	165	110	-50	5	30	35	15	-30	-70
5	10	220	155	-60	10	40	45	20	-50	-100
6	12.5	290	210	-70	100	40	130	25	-60	-125
7	15	360	370	-60	470	725	380	30	-45	-135
8	18	745	610	-50	600	995	3300	55	-40	-125
9	20	1180	665	750	725	1225		150	-20	-130
11	24	2240	855	875	970	1505		625	-15	-150
12	26	2840	945	940	1110	1630		760	- 5	-160
14	30	4635	1120	965	1650	1820		1245	10	-185
15	32	5810	1225	835	1725	1910		1465	25	-190
17	34	8900	1340	700	1760	2040		1800	35	-200
19	36	13160	1430	535	1790	2100		4100	45	-205
21	37	14300	1520	405	1770	2050		8200	65	-210
22	38		1535	330	1765	2035		8930	70	-205
23	0	11950	-810	250	-20	210	590	7690	140	50
24	20		0	250	1000	1060			100	-105
25	30		1265	295	1535	1595			80	-135

Inc. No.	Load per Jack	Strain Gage Number									
		23	24	29	30	31	32	37	38	39	40
1	0.17	0	0	0	0	0	0	0	0	0	0
2	2.5	- 5	15	-65	0	-20	-50	-20	85	95	30
3	5	-15	30	-120	20	-35	-110	-50	180	220	50
4	7.5	-15	40	-175	55	-45	-165	-80	320	310	80
5	10	-20	40	-235	100	-45	-240	-125	555	395	120
6	12.5	-15	35	-300	170	-30	-310	-150	780	525	170
7	15	295	45	-370	305	70	-390	-165	995	620	360
8	18	640	530	-435	405	190	-440	-165	1245	910	520
9	20	720	615	-480	410	280	-585	-130	1400	1090	660
11	24	910	735	-585	520	355	-600	-100	1730	1400	860
12	26	1000	790	-660	590	390	-570	-65	1870	1590	950
14	30	1170	910	-990	770	520	-210	60	2030	2045	1100
15	32	1270	970	-1135	820	620	-105	105	2110	2230	1170
17	34	1405	1010	-1350	915	750	115	120	2210	2430	1255
19	36	1535	1040	-1525	990	875	455	115	2300	2625	1330
21	37	1595	1060	-1605	1030	1040	870	45	2390	2710	1385
22	38	1585	1055	-1590	1030	1055	925	40	2390	2700	1395
23	0	470	150	-80	130	310	910	150	410	80	-10
24	20	1065	630	-1010	510	710	945	170	1580	1500	645
25	30	1340	900	-1600	805	945	970	40	1950	2270	1130

TABLE B.18.2 CENTERLINE STRAIN DISTRIBUTION AND BEAM DEFLECTIONS

Inc.	Strain (in/in x 10^5)							
	1	2	3	5	6	7	8	9
i	0	0	0	0	0	0	0	0
ii	105	83	58	9	- 1	- 1	0	- 4
1	187	159	118	53	33	33	28	27
2	183	157	117	55	40	39	35	32
3	173	150	113	58	44	43	40	40
4	166	143	109	60	48	48	45	44
5	154	136	103	62	53	54	51	50
6	136	130	100	63	58	59	57	57
7	132	120	93	64	63	67	63	63
8	78	90	81	66	70	73	71	72
9	11	50	66	62	81	83	83	84
10	-136	-11	39	57	91	91	92	93
11	-193	-60	8	54	104	104	104	105
12	-369	-126	-30	44	114	116	117	116
13	-413	-221	-70	40	126	127	129	130
14	-568	-336	-126	37	138	139	142	142
15	-759	-474	-191	36	155	157	165	161
16	-917	-582	-243	33	166	169	174	171
17	-1347	-855	-451	25	189	196	200	201
18		-1205	-496	9	220	229	235	236

(i) Before Release; (ii) After Release; (-) Tension

Inc.	Load (kips)	Deflection (in)		
		North	℄	South
1	0.17	0.00	0.00	0.00
2	2.5	.04	.04	.03
3	5.0	.07	.07	.06
4	7.5	.10	.11	.09
5	10.0	.14	.16	.12
6	12.5	.17	.21	.16
7	15.0	.23	.26	.21
8	18.0	.31	.36	.29
9	20.0	.40	.51	.39
11	24.0	.67	.88	.65
12	26.0	.81	1.06	.78
14	30.0	1.24	1.61	1.20
15	32.0	1.49	1.91	1.43
17	34.0	2.10	2.89	2.10
19	36.0	3.36	4.84	3.39
21	37.0	4.34	6.26	4.54
22	38.0	5.17	7.71	5.26
23	0.17	2.71	4.24	3.82
24	20.0	4.00	6.03	4.08
25	30.0	4.75	7.11	4.82
26	38.0		9.16	

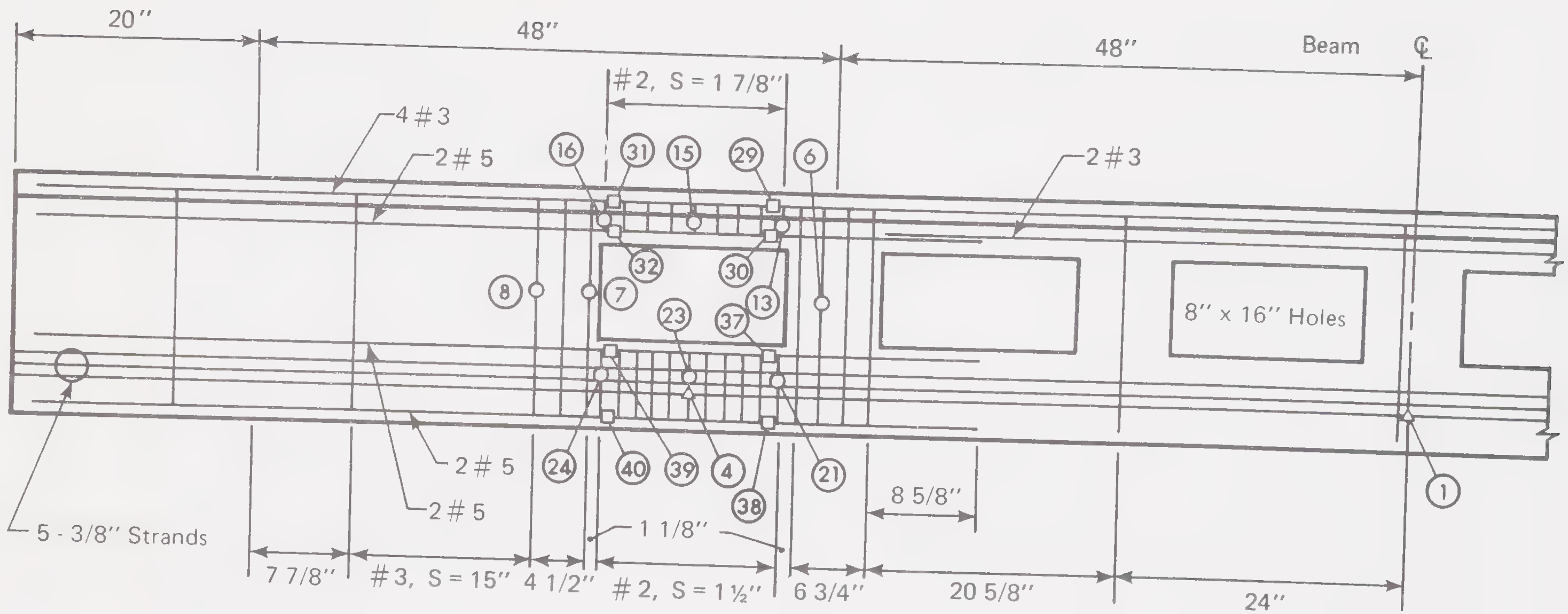


FIGURE B.18. Reinforcement Details and Strain Gage Locations for Beam 18–16–4

TABLE B.19.1 ELECTRICAL STRAIN GAGE MEASUREMENTS (MICRO INCHES PER INCH)

BEAM NO. 19-16-6

Inc. No.	Load per Jack	Strain Gage Number							
		1	3	4	5	6	7	8	9
1	0.15	0	0	0	0	0	0	0	0
2	2		45	20	0	5	20	0	15
3	4		95	55	- 5	30	50	- 5	40
4	6		155	85	- 5	310	200	- 5	60
5	8		220	120	- 5	500	355	0	70
6	10		310	150	5	680	630	70	90
7	11		410	180	15	770	750	100	100
8	12		490	195	30	840	845	145	115
9	13		715	215	35	920	920	175	135
10	14		985	240	60	1000	1015	285	175
11	16		1380	320	70	1175	1160	430	295
12	18		1705	390	55	1310	1335	470	370
13	20		2160	850	30	1450	1575	540	610
14	22		2970	1250	0	1645	1915	620	700
15	23		3560	1455	-30	1780	1915	650	735
16	23		3810	1516	-45	1875	1950	675	730
17	24		3930						

Inc. No.	Load per Jack	Strain Gage Number							
		11	12	13	15	16	17	19	20
1	0.15	0	0	0	0	0	0	0	0
2	2	0	-10	15	0	-10	-15	0	
3	4	10	-25	40	0	-20	-35	- 5	
4	6	10	-50	40	5	-35	-65	-10	
5	8	15	-75	50	10	-50	-75	-20	
6	10	25	-110	40	15	-70	-115	-40	
7	11	30	-135	40	15	-80	-140	-30	
8	12	30	-155	50	20	-85	-155	- 5	
9	13	35	-180	70	20	-90	-150	75	
10	14	40	-205	95	20	-100	-160	180	
11	16	55	-255	185	30	-95	-150	280	
12	18	65	-300	310	40	-100	-135	345	
13	20	130	-365	590	80	-100	-20	425	
14	22	390	-425	720	115	-100	200	465	
15	23	595	-460	775	130	-100	290	480	
16	23	690	-470	770	130	-85	285	450	
17	24								

Continued:-

TABLE B.19.1 ELECTRICAL STRAIN GAGE MEASUREMENTS (MICRO INCHES PER INCH)

BEAM NO. 19-16-6

Inc. No.	Load per Jack	Strain Gage Number							
		21	23	24	25	26	27	30	31
1		0	0	0	0	0	0	0	0
2	2	-15	0	5	-35	15	-15	15	- 5
3	4	-40	- 5	10	-90	40	-40	35	-20
4	6	-70	-15	10	-150	100	-60	30	-30
5	8	-95	-20	5	-200	160	-80	35	-40
6	10	-130	-25	0	-270	285	-105	30	-40
7	11	-155	-30	0	-310	390	-110	35	-40
8	12	-170	-35	0	-340	500	-120	45	-35
9	13	-200	-40	0	-370	610	-120	55	-35
10	14	-215	-45	15	-590	780	-125	90	-20
11	16	-235	-50	40	-445	1130	-120	290	40
12	18	-210	10	120	-460	1420	-120	455	100
13	20	-195	760	325	-540	1920	-105	615	200
14	22	-180	860	485	-400	2090	-70	630	300
15	23	-170	970	555	-350	2135	-20	610	355
16	23	-175	1000	565	-340	2180	+30	585	435
17	24								

Inc. No.	Load per Jack	Strain Gage Number							
		32	33	34	37	38	39	40	41
1	0.15	0	0	0	0	0	0	0	0
2	2	-32	5	75	0	45	15	5	10
3	4	-95	15	190	10	110	120	15	30
4	6	-175	15	330	20	180	220	10	250
5	8	-250	10	520	30	250	310	20	365
6	10	-355	15	870	25	350	485	30	475
7	11	-415	50	1275	25	425	610	40	545
8	12	-465	95	1500	20	535	700	45	590
9	13	-530	170	1750	10	755	835	50	630
10	14	-610	310	2065	0	985	1050	70	690
11	16	-760	600	2640	30	1375	1325	100	820
12	18	-905	830	3110	210	1740	1600	140	945
13	20	-2235	1295	17000	445	2225	2195	255	1065
14	22	-4205	1870	22890	680	2805	2640	570	1250
15	23	-5325	2275	23800	800	3280	2945	700	1350
16	23	-5900	2440	24375	860	3225	2970	770	1480
17	24								

TABLE B.19.2 CENTERLINE STRAIN DISTRIBUTION AND BEAM DEFLECTIONS

Inc.	Strain (in/in x 10^5)							
	1	2	3	5	6	7	8	9
i	0	0	0	0	0	0	0	0
ii	81	70	50	78	1	0	- 1	- 3
1	175	153	114	52	33	31	21	27
2	173	152	114	54	38	37	25	31
3	162	144	108	57	45	46	32	38
4	150	137	103	59	53	52	39	45
5	139	128	97	61	58	58	46	51
6	125	118	90	63	65	66	54	61
7	115	109	86	64	70	70	59	65
8	83	94	79	64	74	75	64	71
9	37	69	68	64	79	80	70	76
10	-25	28	40	58	92	93	84	89
11	-118	-35	-12	46	112	115	105	109
12	-225	-100	-66	38	129	131	122	126
13	-345	-205	-153	28	154	157	147	153
14	-595	-404	-809	-39	222	227	230	224
15		-1154	-1364	-233	333	342	336	343
16								
17								

(i) Before Release; (ii) After Release; (-) Tension

Inc.	Load (kips)	Deflection (in)		
		North	℄	South
1	0.15	0.00	0.00	0.00
2	2	.02	.03	.02
3	4	.06	.08	.06
4	6	.11	.13	.11
5	8	.16	.18	.16
6	10	.23	.25	.23
7	11	.28	.30	.28
8	12	.32	.34	.32
9	13	.37	.40	.37
10	14	.49	.53	.49
11	16	.73	.80	.73
12	18	.96	1.04	.96
13	20	1.36	1.48	1.36
14	22	2.22	2.43	2.24
15	23	3.76	4.17	3.76
16	23	4.69	5.27	4.60
17	24			

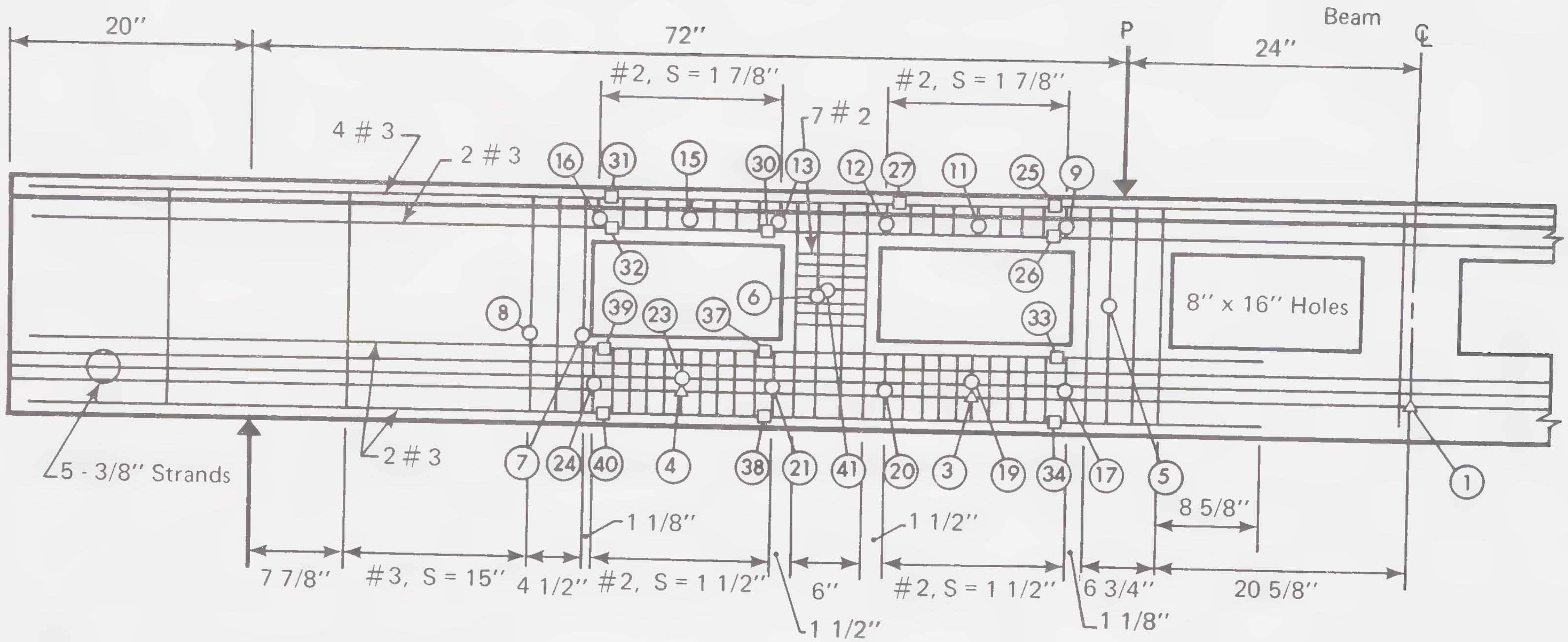


FIGURE B.19. Reinforcement Details and Strain Gage Locations for Beam 19–16–6

TABLE B.20.1 ELECTRICAL STRAIN GAGE MEASUREMENTS (MICRO INCHES PER INCH)

BEAM NO. 20-26-5

Inc. No.	Load per Jack	Strain Gage Number								
		1	4	6	7	8	13	15	16	23
1	0.15	0	0	0	0	0	0	0	0	0
2	1	25	15	0	15	0	15	0	0	0
3	2	50	25	0	40	0	35	0	0	0
4	3	80	45	0	90	- 5	95	0	- 5	- 5
5	4	110	60	0	170	0	160	5	-10	-10
7	6	175	100	0	370	20	165	10	-20	-15
9	8	230	135	5	480	80	150	10	-30	-25
11	10	305	180	15	680	275	155	15	-50	-30
12	11	355	205	10	805	390	165	15	-60	-35
13	12	400	235	5	945	435	170	20	-60	-35
14	13	470	255	0	1060	470	185	20	-65	-30
15	14	640	285	5	1220	530	210	20	-70	-30
16	15	855	300	10	1315	570	250	25	-85	-20
17	16	1235	335	10	1440	640	350	30	-90	55
18	17	1565	365	20	1540	695	410	40	-90	210
19	18	1880	720	20	1670	760	480	40	-90	300
20	19	2270	875	590	1845	850	700	50	-70	375
21	20	2660	1005	710	1950	920	825	60	-25	435
22	21	3240	1320	825	2080	1025	1390	80	60	510
23	22								140	550

Inc. No.	Load per Jack	Strain Gage Number								
		24	29	30	31	32	37	38	39	40
1	0.15	0	0	0	0	0	0	0	0	0
2	1	-35	-10	20	- 5	-30	- 5	40	30	0
3	2	-20	-25	45	-10	-70	-30	85	65	- 5
4	3	25	-35	80	-20	-120	-30	150	115	-10
5	4	135	-50	115	-30	-175	-50	215	170	-15
7	6		-75	205	-50	-310	-100	375	390	-30
9	8		-100	340	-50	-460	-160	615	685	-40
11	10		-125	570	0	-700	-225	1090	1005	-60
12	11		-135	715	80	-870	-250	1410	1285	-75
13	12		-130	860	115	-990	-255	1675	1465	-90
14	13		-125	1010	170	-1155	-250	1975	1680	-90
15	14		-125	1250	250	-1320	-210	2330	1920	-80
16	15		-110	1445	295	-1085	-205	2660	2110	-65
17	16		-60	1710	350	-1100	-165	2995	2375	-50
18	17		-35	1895	395	-1260	-140	3300	2660	15
19	18		-20	1950	440	-1695	-55	3260	2840	45
20	19		25	1085	600	-3000	'100	7250	3010	55
21	20		75	2045	730	-4500	215	19900	3090	90
22	21		365	2120	920	-14000	320	22900	3060	185
23	22		1010	2205	1110		200		3050	250

TABLE B.20.2 CENTERLINE STRAIN DISTRIBUTION AND BEAM DEFLECTIONS

Inc.	Strain (in/in x 10^5)							
	1	2	3	5	6	7	8	9
i	0	0	0	0	0	0	0	0
ii	90	71	45	12	2	3	0	- 2
1	209	171	130	60	48	41	33	22
2	210	172	131	61	50	43	35	24
3	207	171	130	61	52	46	38	27
4	204	168	128	61	54	48	41	31
5	199	163	126	63	56	51	44	34
7	189	156	121	64	62	57	50	42
9	181	149	117	65	66	62	55	49
11	170	142	112	69	72	68	62	57
12	163	137	108	68	76	71	65	62
13	157	133	107	71	78	74	69	64
14	136	123	103	72	81	77	72	68
15	92	102	95	71	87	83	78	75
16	44	78	85	69	92	88	83	81
17	-18	32	59	64	102	98	94	93
18	-66	- 2	40	61	108	104	100	98
19	-109	-33	20	59	116	112	109	108
20	-168	-79	- 8	55	127	122	120	119
21								
22								
23								

(i) Before Release; (ii) After Release; (-) Tension

Inc.	Load (kips)	Deflection (in)		
		North	℄	South
1	0.15	0.00	0.00	0.00
2	1	.01	.01	.01
3	2	.03	.03	.03
4	3	.05	.05	.05
5	4	.06	.07	.06
7	6	.11	.12	.11
9	8	.15	.17	.15
11	10	.22	.24	.22
12	11	.27	.29	.27
13	12	.31	.34	.31
14	13	.36	.39	.36
15	14	.44	.48	.44
16	15	.51	.55	.51
17	16	.64	.71	.64
18	17	.73	.82	.73
19	18	.84	.94	.84
20	19	1.02	1.15	1.02
21	20	1.21	1.35	1.21
22	21	1.51	1.67	1.51
23	22		2.13	

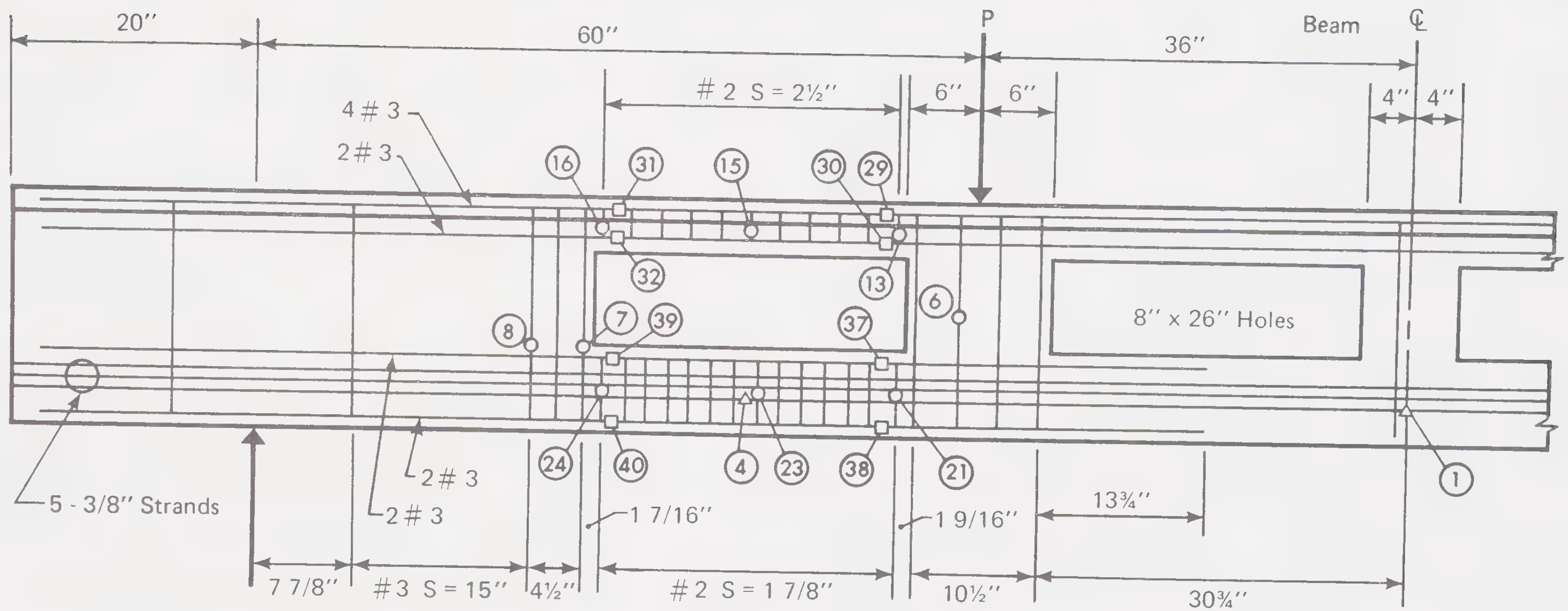


FIGURE B.20. Reinforcement Details and Strain Gage Locations for Beam 20–26–5

TABLE B.21.1 ELECTRICAL STRAIN GAGE MEASUREMENTS (MICRO INCHES PER INCH)

BEAM NO. 21-26-5

Inc. No.	Load per Jack	Strain Gage Numbers								
		1	4	6	7	8	13	15	16	21
1	0.15	0	0	0	0	0	0	0	0	0
2	2	45	25	0	40	0	50	5	- 5	-15
3	4	115	55	- 5	155	15	115	5	-20	-25
4	6	180	95	- 5	365	55	140	10	-40	- 5
5	8	245	135	- 5	560	275	135	15	-55	-20
6	10	330	225	0	765	425	125	15	-70	0
7	11	385	285	0	835	510	135	20	-80	30
8	12	430	460	0	885	565	155	25	-85	-85
9	13.5	600	575	15	1065	590	205	35	-85	215
10	15	900	615	20	1225	625	315	40	-105	200
11	16.5	1685	715	50	1410	680	530	60	-125	225
12	18	2095	805	30	1555	740	630	75	-140	200
13	19.5	2710	915	35	1680	850	900	115	-155	145
14	21	3435	1025	50	1830	950	1185	240	-165	20
15	22.5	4300	1120	590	1935	1040	1845	370	-220	-15
16	24	5580	1225	680	1975	1160	1990	545	-275	-80
18	26	11900	1360	640	1995	1345	2300	1345	-255	-15
19	27	16650	1480	580	1990	1420	2440	1800	-175	350
20	28	19900	1610	540	1965	1475		1890	-45	1025
21	29	24000								

Inc. No.	Load per Jack	Strain Gage Number									
		23	24	29	30	31	32	37	38	39	40
1	0.15	0	0	0	0	0	0	0	0	0	0
2	2	- 5	10	-30	30	-10	-55	-15	65	75	0
3	4	- 5	35	-80	90	-25	-150	-35	210	250	0
4	6	- 5	75	-130	185	-25	-265	-65	400	415	5
5	8	5	125	-175	290	- 5	-375	-85	630	575	15
6	10	140	240	-230	425	40	-525	-85	940	820	25
7	11	270	410	-265	540	85	-615	-45	1130	955	40
8	12	630	505	-295	635	130	-690	-15	1240	1045	55
9	13.5	785	590	-325	810	220	-795	40	1550	1185	105
10	15	905	695	-355	920	290	-865	105	1590	1365	135
11	16.5	1055	795	-340	1045	370	-960	195	2020	1585	180
12	18	1175	865	-345	1125	435	-1435	285	2215	1775	230
13	19.5	1325	935	-335	1275	500	-2410	390	2440	1990	310
14	21	1470	1005	-365	1440	560	-4355	520	2650	2225	365
15	22.5	1590	1060	-390	1625	635	-6030	680	2830	2485	405
16	24	1665	1125	-420	1815	695	-8050	810	3100	2730	450
18	26	1630	1190	-675	2350	880	-11575	1245	15960	3120	505
19	27	1650	1225	-695	2395	985	-12150	1395	17250	3720	510
20	28	1685	1060		2465	1255	-13070	1425	18780	14600	530
21	29										

TABLE B.21.2 CENTERLINE STRAIN DISTRIBUTION AND BEAM DEFLECTIONS

Inc.	Strain (in/in x 10^5)							
	1	2	3	5	6	7	8	9
i	0	0	0	0	0	0	0	0
ii	100	77	56	9	- 4	- 2	- 3	- 6
1	212	174	130	57	34	32	35	22
2	210	172	130	61	39	37	39	27
3	203	167	127	63	44	43	45	34
4	190	156	120	65	50	50	52	39
5	184	153	117	68	54	55	58	46
6	173	144	112	71	60	63	65	54
7	165	137	108	72	64	67	69	59
8	157	131	104	72	67	71	73	63
9	148	122	100	73	73	77	80	70
10	113	101	82	69	85	89	92	83
11	40	48	5	66	96	100	102	95
12	0	17	-30	62	105	110	112	105
13	-63	-28	-70	60	116	120	121	115
14	-142	-82	-110	57	126	130	131	127
15	-233	-144	-150	56	140	143	144	143
16	-350	-230	-217	54	161	165	165	164
18	-803	-604	-617	- 7	234	239	240	242
19	-1181	-912	-969	-70	280	285	287	257
20	-1403	-1092	-1077	-97	303	308	309	312
21								
(i) Before Release; (ii) After Release; (-) Tension								

Inc.	Load (kips)	Deflection (in)		
		North	℄	South
1	0.15	0.00	0.00	0.00
2	2	.02	.02	.02
3	4	.07	.07	.07
4	6	.11	.12	.11
5	8	.16	.17	.16
6	10	.22	.24	.22
7	11	.27	.29	.26
8	12	.31	.33	.30
9	13.5	.37	.40	.37
10	15	.46	.52	.46
11	16.5	.60	.68	.59
12	18	.72	.82	.71
13	19.5	.86	.98	.85
14	21	1.04	1.19	1.02
15	22.5	1.26	1.45	1.24
16	24	1.54	1.78	1.52
18	26	2.58	3.06	2.52
19	27	3.30	4.00	3.21
20	28	3.88	4.69	3.76
21	29		5.19	

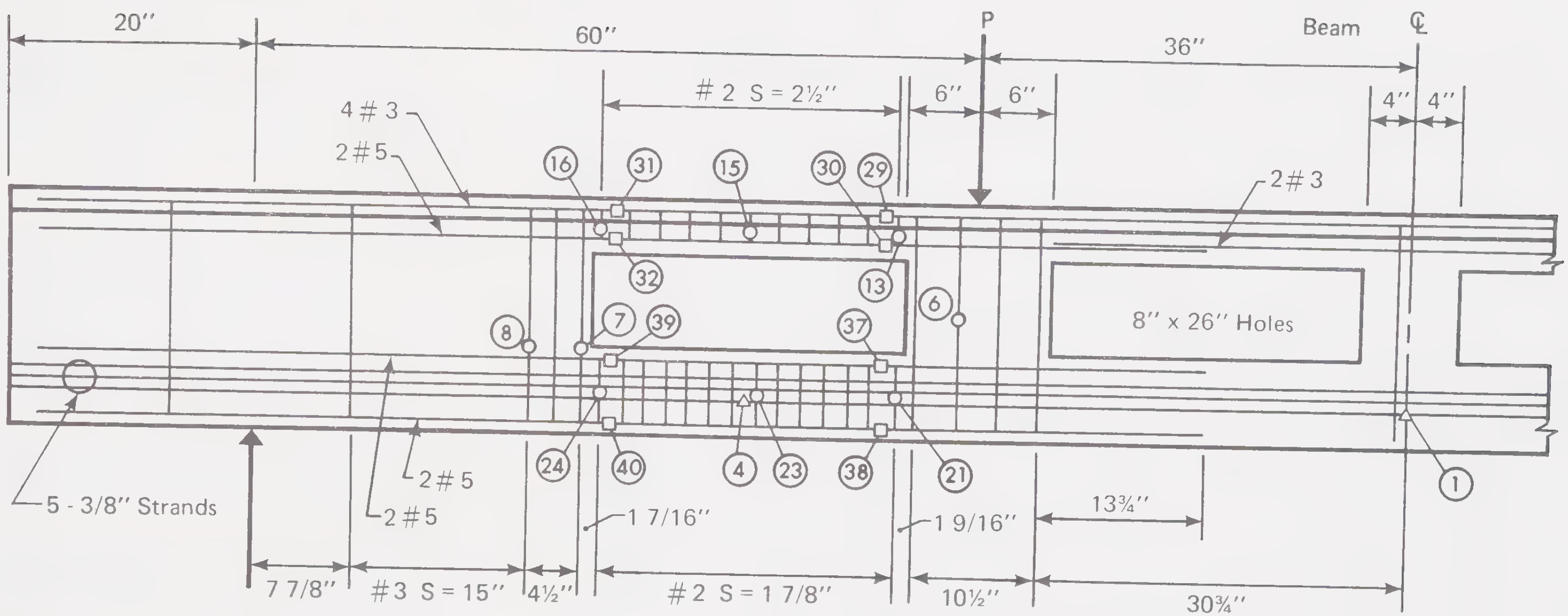


FIGURE B.21. Reinforcement Details and Strain Gage Locations for Beam 21–26–5

TABLE B.22.1 ELECTRICAL STRAIN GAGE MEASUREMENTS (MICRO INCHES PER INCH)

BEAM NO. 22-26-5

Inc. No.	Load per Jack	Strain Gage Number								
		1	4	6	7	8	13	15	16	21
1	0.15	0	0	0	0	0	0	0	0	0
2	1	25	20	5	20	10	10	5	0	0
3	2	50	30	5	40	5	30	10	0	-10
4	3	85	50	5	100	5	60	10	-10	-20
5	4	120	70	5	295	10	80	15	-15	-25
6	5	150	85	5	440	20	85	15	-20	-40
7	6	185	105	5	585	40	90	15	-30	-45
8	7	220	130	5	675	55	90	20	-35	-55
9	8	250	145	10	760	100	85	20	-40	-70
10	9	300	170	15	870	155	85	25	-45	-90
12	11	390	220	30	1170	230	90	30	-45	-80
13	12	440	245	25	1295	255	100	30	-55	-70
14	13	495	270	90	1460	290	115	40	-60	-50
15	14	830	330	135	1675	320	160	40	-50	-35
16	15	1115	595	165	1820	350	220	45	-40	-5
17	17	1820	705	225	2040	420	430	60	-20	45
18	19	2520	780	280	7250	570	640	75	60	305
19	20	2800	940	300	14100	650	665	80	115	270
20	21	3550	1480	305		785	655	80	230	290
21	22						750			

Inc. No.	Load per Jack	Strain Gage Numbers									
		23	24	29	30	31	32	37	38	39	40
1	0.15	0	0	0	0	0	0	0	0	0	0
2	1	5		-10	15	0	-20	-15	40	30	
3	2	0		-25	50	-5	-55	-25	90	75	
4	3	0		-50	100	-10	-100	-55	150	130	
5	4	-5		-70	120	-15	-150	-80	225	190	
6	5	-10		-90	240	-15	-200	-110	310	260	
7	6	-15		-105	320	-20	-260	-140	400	340	
8	7	-20		-125	400	-20	-320	-180	535	445	
9	8	-20		-145	530	-10	-390	-225	700	630	
10	9	-25		-170	705	40	-510	-290	960	860	
12	11	-40		-210	1100	210	-740	-660	1465	1210	
13	12	-40		-225	1270	295	-845	-1010	1680	1370	
14	13	-30		-235	1400	410	-950	-1260	1920	1635	
15	14	30		-235	1700	550	-970	-1420	2220	1860	
16	15	700		-220	1915	640	-990	-1610	2470	2070	
17	17	1240		-120	2250	880	-1105	-1940	3140	2535	
18	19	1860		190	2215	1285	-3220	-3150	3450	3135	
19	20	2090		390	8700	1575	-4210	-3900	3440	3580	
20	21	2275		855	20420	2360	-7300	-9000	3550	6200	
21	22										

TABLE B.22.2 CENTERLINE STRAIN DISTRIBUTION AND BEAM DEFLECTIONS

Inc.	Strain (in/in x 10^5)							
	1	2	3	5	6	7	8	9
i	0	0	0	0	0	0	0	0
ii	108	87	59	10	- 5	- 1	0	- 4
1	222	184	136	49	30	- 2	25	24
2	222	185	136	50	32	- 1	28	26
3	218	182	134	52	33	1	29	27
4	213	180	132	53	36	4	32	30
5	208	174	128	54	37	7	36	34
6	203	170	127	56	40	10	39	37
7	198	166	123	56	43	13	42	41
9	189	158	118	58	47	19	48	47
10	182	154	114	58	49	22	51	51
11	176	149	112	60	53	25	54	54
12	167	144	109	61	56	27	57	58
13	161	138	105	62	58	31	61	62
14	152	132	103	64	62	35	64	67
15	108	109	91	66	68	40	70	73
16	67	82	76	67	72	44	74	77
17	-16	- 3	25	56	88	59	89	84
18	-93	-83	-28	47	105	75	106	111
19	-122	-120	-53	46	112	82	112	118
20	-274	-182	-94	40	121	92	123	129
21								

(i) Before Release; (ii) After Release; (-) Tension

Inc.	Load (kips)	Deflection (in)		
		North	℄	South
1	0.15	0.00	0.00	0.00
2	1	.02	.02	.02
3	2	.04	.03	.03
4	3	.06	.06	.05
5	4	.08	.08	.07
6	5	.10	.11	.10
7	6	.13	.13	.12
9	8	.17	.18	.17
10	9	.21	.22	.20
11	10	.24	.26	.24
12	11	.29	.31	.28
13	12	.34	.36	.33
14	13	.39	.41	.37
15	14	.47	.50	.46
16	15	.53	.57	.52
17	17	.78	.85	.76
18	19	1.09	1.19	1.07
19	20	1.29	1.38	1.24
20	21	1.70	1.89	1.61
21	22		2.21	

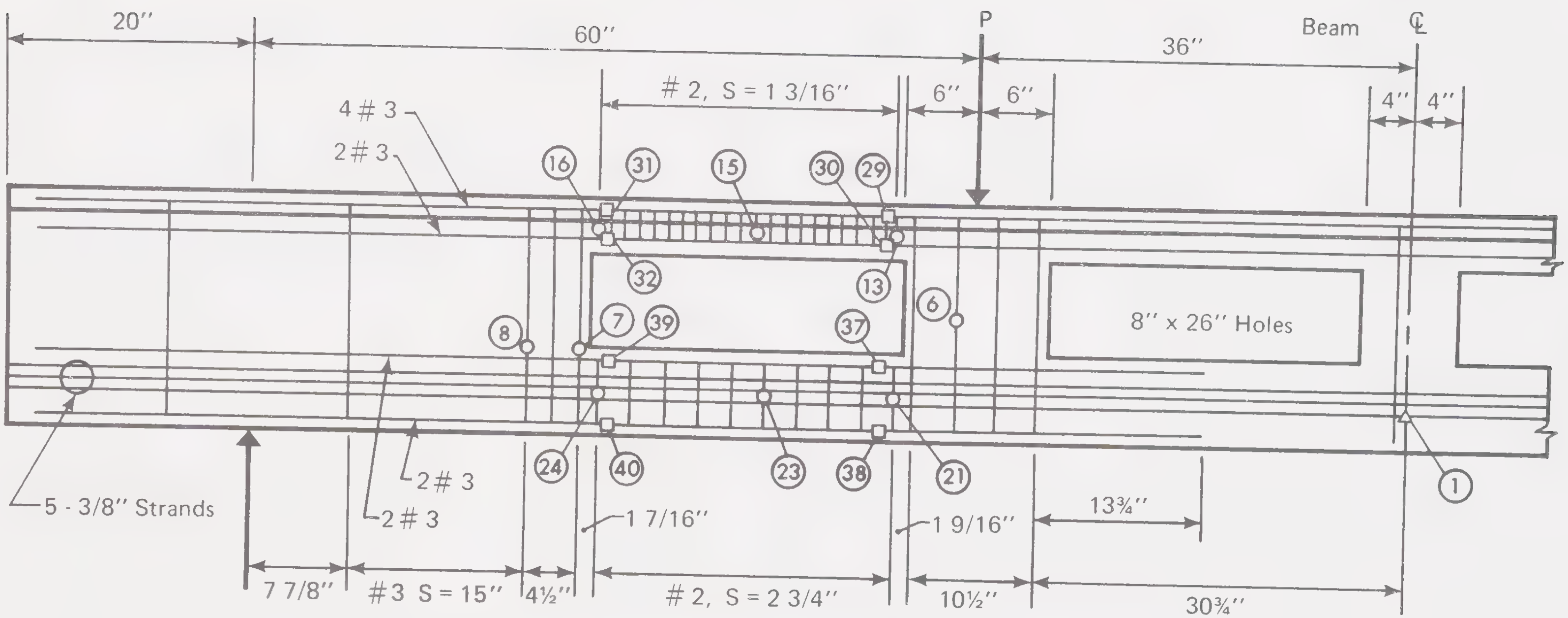


FIGURE B.22. Reinforcement Details and Strain Gage Locations for Beam 22–26–5

TABLE B.23.1 ELECTRICAL STRAIN GAGE MEASUREMENTS (MICRO INCHES PER INCH)

BEAM NO. 23-16-4

Inc. No.	Load per Jack	Strain Gage Numbers								
		1	4	6	7	8	8A	13	23	30
1	.169	0	0	0	0	0	0	0	0	0
2	2.5	65	35	-10	10	10	0	10	0	30
3	5	130	65	-25	20	25	- 5	15	0	50
4	7.5	185	95	-35	70	35	-10	10	- 5	65
5	10	250	130	-35	190	25	-10	5	-10	90
6	12.5	320	170	-25	190	35	-10	30	-15	135
7	15	410	215	-10	235	135	- 5	150	35	215
8	17	530	385	870	480	870	35	285	470	330
9	19	765	480	1490	540	980	45	410	595	590
10	20	1070	535	1540	575	1030	45	425	640	655
11	22	1570	610	1595	655	1125	50	465	710	850
12	24	2130	700	1625	790	1235	50	510	800	970
13	26	2775	785	1675	940	1400	45	535	900	1150
14	28	3600	870		1135	1535	35	545	990	1400
15	30	4700	950		1320	1655	25	565	1070	1600
16	32	6000	1045		1435	1725	25	615	1145	1800
17	34	8900	1150		1490	1755	30	675	1220	1995
18	36	15000	1245		1540	1785	35	740	1280	2155
19	38		1355		1600	1775	35	785	1345	2215

Inc. No.	Load per Jack	Strain Gage Numbers								
		32	38	39						
1	.169	0	0	0						
2	2.5	-65	80	50						
3	5	-130	160	95						
4	7.5	-195	240	150						
5	10	-225	380	230						
6	12.5	-380	595	385						
7	15	-500	790	540						
8	17	-660	970	700						
9	19	-775	1115	850						
10	20	-830	1195	900						
11	22	-940	1335	1020						
12	24	-1020	1540	1180						
13	26	-1110	1700	1340						
14	28	-1220	1810	1600						
15	30	-1310	1920	1800						
16	32	-1500	2025	1990						
17	34	-1665	2120	2140						
18	36	-1785	2170	2340						
19	38	-1790	2245	2485						

TABLE B.23.2 CENTERLINE STRAIN DISTRIBUTION AND BEAM DEFLECTIONS

Inc.	Strain (in/in x 10^5)							
	1	2	3	5	6	7	8	9
i	0	0	0	0	0	0	0	0
ii	93	73	50	11	1	0	1	- 3
1	185	153	112	44	23	26	24	21
2	177	147	109	47	28	31	29	27
3	171	144	106	50	34	37	35	33
4	161	135	101	52	39	42	40	38
5	153	129	100	54	45	47	45	44
6	143	122	95	57	51	54	52	49
7	128	113	89	56	57	59	57	54
8	101	98	81	57	62	66	63	57
9	73	71	69	58	71	73	70	67
10	8	41	60	57	77	77	75	73
11	-81	-37	22	48	91	90	87	85
12	-156	-121	-22	44	101	100	97	97
13	-229	-208	-75	35	115	114	110	109
14	-324	-316	-235	31	129	126	122	121
15	-443	-453	-217	20	144	142	138	137
16	-601	-616	-317	9	162	161	156	156
17	-956	-1026	-562	-17	201	197	183	192
18			-1267	-77	278	272	265	264
19					341	325	316	318

(i) Before Release; (ii) After Release; (-) Tension

Inc.	Load (kips)	Deflection (in)		
		North	℄	South
1	0.17	0.00	0.00	0.00
2	2.5	.03	.04	.03
3	5.0	.07	.08	.07
4	7.5	.10	.12	.10
5	10.0	.13	.16	.14
6	12.5	.17	.21	.18
7	15.0	.21	.26	.22
8	17.0	.26	.32	.27
9	19.0	.31	.44	.34
10	20.0	.37	.48	.38
11	22.0	.49	.65	.50
12	24.0	.63	.84	.65
13	26.0	.80	1.06	.80
14	28.0	1.00	1.31	.99
15	30.0	1.22	1.61	1.30
16	32.0	1.53	2.01	1.48
17	34.0	2.13	2.86	2.05
18	36.0	3.52	4.91	3.40
19	38.2	5.22	7.76	5.27

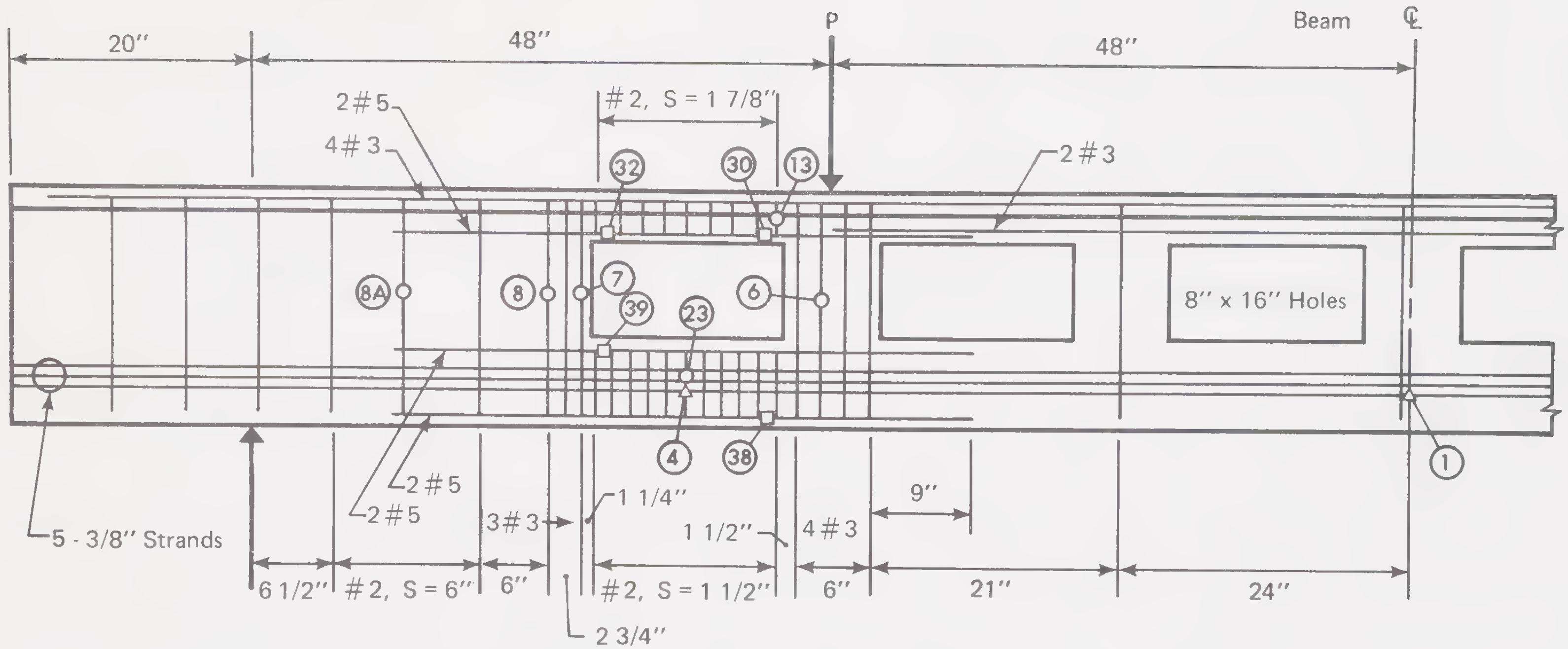


FIGURE B.23. Reinforcement Details and Strain Gage Locations for Beam 23–16–4

TABLE B.24.1 ELECTRICAL STRAIN GAGE MEASUREMENTS (MICRO INCHES PER INCH)

BEAM NO. 24-16-6

Inc. No.	Load per Jack	Strain Gage Number								
		1	3	5	7	8	8A	9	19	26
1	0.15	0	0	0	0	0	0	0	0	0
2	2	50	40	- 5	10	0	0	10	- 5	15
3	4	115	90	-20	25	0	0	20	-15	30
4	6	190	150	-30	25	0	5	45	-25	45
5	8	255	200	-35	15	- 5	5	45	-30	55
6	10	350	270	-40	10	- 5	0	40	-50	80
7	11	405	315	-35	15	- 5	5	45	-55	130
8	12	500	355	-25	25	- 5	5	55	-65	200
9	13	800	385	-15	25	0	5	75	-70	300
10	14	1250	840	5	45	5	5	125	370	465
11	15	1580	975	5	65	5	5	195	485	595
12	16	2075	1165	35	115	5	5	345	595	830
13	18	3000	1565	655	700	35	0	525	715	1255
14	20	4800	2025	535	790	45	0	600	820	1675
15	22	9500	2635	430	895	355	0	670	885	2120
16	23		3210	295	995		15	650	915	2135
17	24		3530	200	1040		21			

Inc. No.	Load per Jack	Strain Gage Numbers								
		28	34	35						
1	0.15	0	0	0						
2	2	-45	80	50						
3	4	-115	185	120						
4	6	-190	305	200						
5	8	-270	435	280						
6	10	-390	890	385						
7	11	-480	1175	475						
8	12	-560	1315	645						
9	13	-640	1485	1055						
10	14	-760	1755	1310						
11	15	-850	1965	1530						
12	16	-975	2275	1920						
13	18	-1380	2750	2270						
14	20	-4320	2920	3185						
15	22	-6130	6150	3335						
16	23	-6580		5000						
17	24									

TABLE B.24.2 CENTERLINE STRAIN DISTRIBUTION AND BEAM DEFLECTIONS

Inc.	Strain (in/in x 10^5)							
	1	2	3	5	6	7	8	9
i	0	0	0	0	0	0	0	0
ii	88	62	41	9	- 2	- 2	- 5	- 4
1	166	129	99	43	27	29	27	23
2	162	127	100	45	33	35	32	28
3	151	120	92	49	38	40	38	30
4	140	112	87	51	45	48	47	42
5	129	104	83	53	51	55	52	49
6	115	94	79	57	59	62	60	57
7	107	90	75	58	60	65	64	61
8	82	77	71	60	65	69	67	66
9	47	57	65	59	69	72	71	70
10	-19	14	43	56	76	80	79	78
11	-57	-14	24	52	83	88	85	86
12	-121	-65	-10	47	95	98	95	98
13	-363	-162	-69	42	112	116	112	117
14	-683	-349	-178	39	140	143	141	145
15	-1599	-814	-468	-25	194	199	195	203
16			-1128	-155	274	276	272	287
17								

(i) Before Release; (ii) After Release; (-) Tension

Inc.	Load (kips)	Deflection (in)		
		North	℄	South
1	0.15	0.00	0.00	0.00
2	2	.03	.02	.02
3	4	.07	.07	.07
4	6	.11	.11	.10
5	8	.15	.15	.14
6	10	.19	.20	.19
7	11	.23	.24	.23
8	12	.26	.28	.26
9	13	.30	.32	.30
10	14	.40	.42	.39
11	15	.49	.53	.49
12	16	.60	.66	.61
13	18	.84	.91	.84
14	20	1.25	1.39	1.26
15	22	2.22	2.50	2.26
16	23	3.98	4.53	4.01
17	24		6.00	

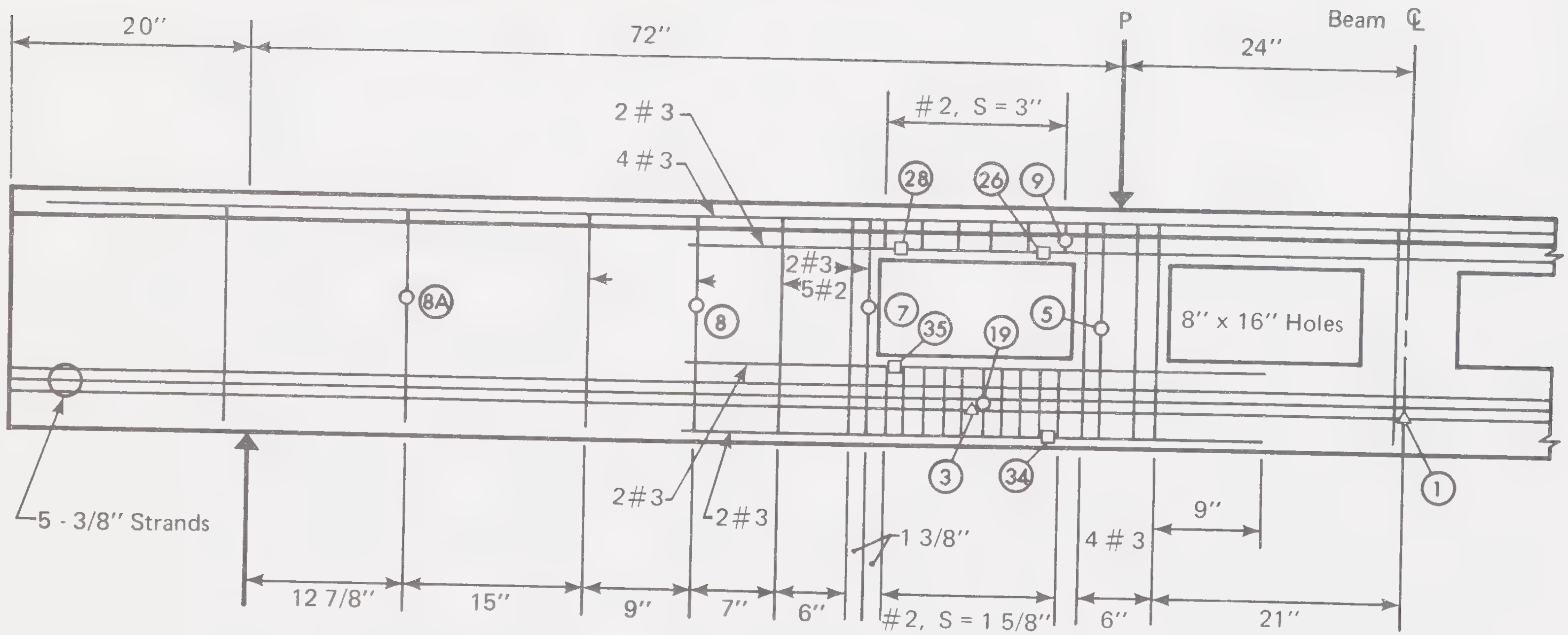


FIGURE B.24. Reinforcement Details and Strain Gage Locations for Beam 24–16–6

TABLE B.25.1 ELECTRICAL STRAIN GAGE MEASUREMENTS (MICRO INCHES PER INCH)

BEAM NO. 25-16-6

Inc. No.	Load per Jack	Strain Gage Numbers								
		1	3	4	5	6	7	9	13	23
1	0.15	0	0	0	0	0	0	0	0	0
2	2	55	45	25	0	0	5	15	15	- 5
3	4	135	100	60	-10	20	10	40	40	-10
4	6	215	165	90	-15	65	25	50	50	-15
5	8	290	225	125	-25	420	55	30	60	-20
6	10	390	305	165	-25	610	55	15	65	-30
7	11	460	360	190	-15	710	100	10	85	-30
8	12	595	400	210	- 5	785	150	5	105	-35
9	13	745	580	230	10	865	270	5	130	-40
10	14	1190	945	260	30	950	405	35	165	-50
11	15	1645	1175	285	55	1050	550	80	200	-60
12	16	2100	1560	320	70	1165	695	185	230	-43
13	18	3090	2355	415	85	1270	930	350	410	-25
14	20	4745	3630	970	650	1440	1130	660	530	440
15	22				750					

Inc. No.	Load per Jack	Strain Gage Numbers								
		26	28	30	32	34	35	38	39	41
1	0.16	0	0	0	0	0	0	0	0	0
2	2	30	-65	20	-50	75	40	45	45	0
3	4	75	-165	50	-125	195	105	115	115	- 5
4	6	110	-285	70	-210	330	170	195	205	- 5
5	8	135	-395	50	-320	480	225	255	320	150
6	10	200	-575	50	-525	960	405	360	490	220
7	11	340	-590	55	-630	1420	545	420	585	265
8	12	470	-605	55	-710	1780	715	475	660	290
9	13	635	-795	65	-830	2050	860	550	730	330
10	14	890	-1015	70	-1090	2630	1165	875	910	395
11	15	1095	-1230	80	-1375	3300	1390	1170	1030	450
12	16	1315	-1665	80	-2150	4150	1765	1668	1370	515
13	18	1350	-2475	180	-3300	5665	2445	2165	1710	650
14	20	1610	-4200	295	-5350	11750	5850	8600	2330	765
15	22									

TABLE B.25.2 CENTERLINE STRAIN DISTRIBUTION AND BEAM DEFLECTIONS

Inc.	Strain (in/in x 10^5)							
	1	2	3	5	6	7	8	9
i	0	0	0	0	0	0	0	0
ii	90	77	46	18	- 3	- 1	- 1	- 5
1	155	131	84	36	13	15	16	10
2	149	127	82	41	17	19	20	14
3	139	118	76	42	23	26	28	23
4	129	111	73	43	31	33	35	31
5	116	101	66	47	36	41	41	37
6	102	91	62	49	44	47	50	45
7	89	84	56	50	46	52	54	49
8	64	69	50	51	53	57	58	54
9	42	54	42	52	56	62	63	69
10	-53	-12	17	49	66	71	73	70
11	-148	-80	7	46	74	79	82	79
12	-273	-182	-36	41	86	90	91	89
13	-378	-355	-101	31	104	107	111	108
14	-458	-636	-218	20	131	135	137	138
15								

(i) Before Release; (ii) After Release; (-) Tension

Inc.	Load (kips)	Deflection (in)		
		North	℄	South
1	0.15	0.00	0.00	0.00
2	2	.03	.04	.02
3	4	.07	.08	.06
4	6	.12	.13	.11
5	8	.17	.18	.15
6	10	.23	.25	.23
7	11	.28	.30	.27
8	12	.31	.34	.30
9	13	.36	.39	.35
10	14	.47	.50	.45
11	15	.57	.62	.55
12	16	.76	.81	.73
13	18	1.05	1.11	1.02
14	20	1.62	1.68	1.60
15	22		2.39	

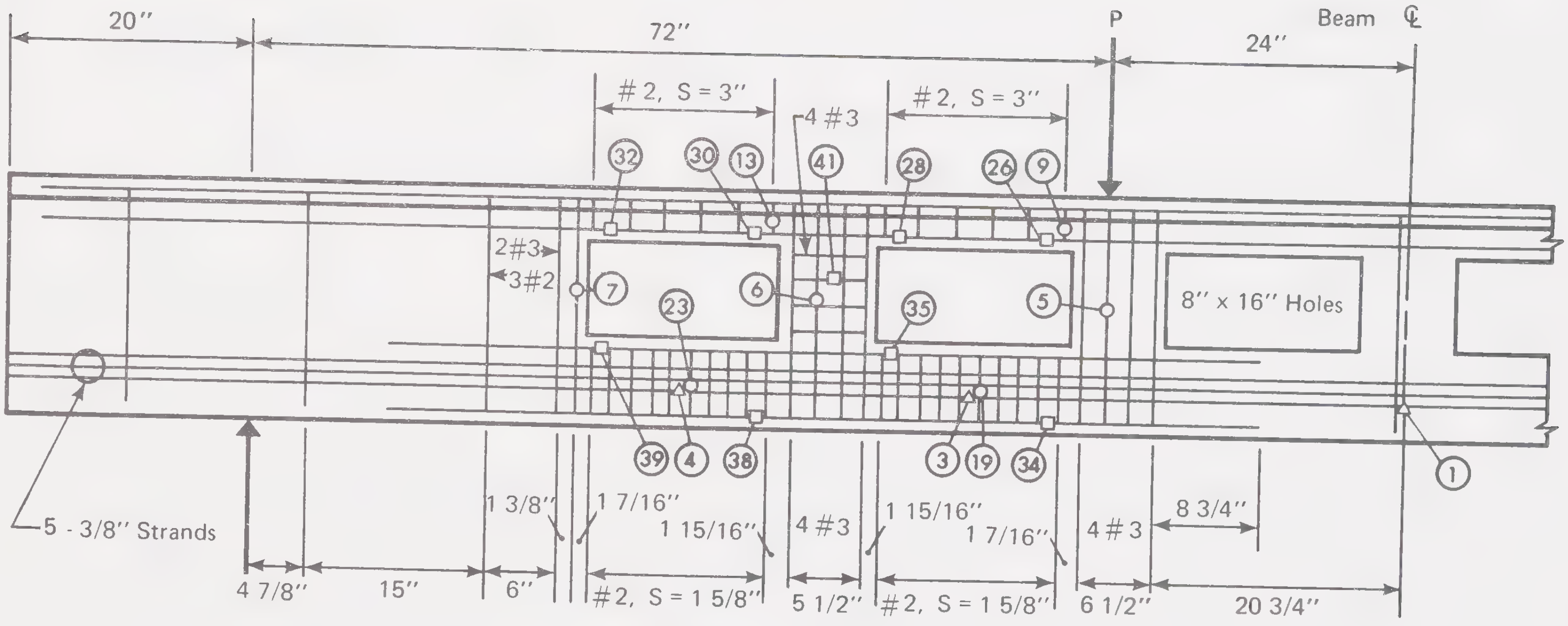


FIGURE B.25. Reinforcement Details and Strain Gage Locations for Beam 25–16–6

TABLE B.26.1 ELECTRICAL STRAIN GAGE MEASUREMENTS (MICRO INCHES PER INCH)

BEAN NO. 26-21-7

Inc. No.	Load per Jack	Strain Gage Numbers									
		1	3	4	5	6	7	8	9	13	19
1	.15	0	0	0	0	0	0	0	0	0	0
2	2	60	40	20	-10	- 5	5	0	10	- 5	-15
3	4	150	100	55	-20	10	20	0	60	80	-30
4	6	245	175	85	-25	245	10	5	145	95	-50
5	8	335	240	120	-25	400	25	20	130	95	-80
6	9	400	305	140	-20	535	135	30	130	165	-65
7	10	445	340	155	-15	600	165	30	130	210	-20
8	11	525	390	175	-10	680	270	25	115	230	80
9	12	1510	610	195	-10	755	480	30	115	585	165
10	13	2025	870	215	-20	830	590	30	130	1040	225
11	14	2600	1090	245	680	920	710	30	150	1390	305
12	15	3210	1240	260	745	995	820	25	180		350
13	16	4250	1460	290	810	1120	1010	25	260		395
14	17	5235	1650	315	845	1230	1155	20	350		410
15	18	6700	1850	545	870	1360	1280	35	410		430
16	19	3630	2450	840	790	1650	1450	130	435		590
17	20		2820	950	620	1910	1600	280	490		

Inc. No.	Load per Jack	Strain Gage Numbers									
		23	26	28	30	32	34	35	38	39	41
1	.15	0	0	0	0	0	0	0	0	0	0
2	2		10	-65	15	-60	90	55	60	50	0
3	4		50	-175	45	-155	240	145	150	140	0
4	6		145	-300	60	-270	430	245	250	260	280
5	8		285	-440	75	-395	825	440	350	370	385
6	9		430	-560	100	-490	1160	625	440	470	450
7	10		555	-655	120	-575	1470	790	600	600	475
8	11		785	-800	170	-690	1885	1120	830	780	540
9	12		985	-915	225	-800	2160	1375	1000	900	575
10	13		1180	-1030	300	-900	2460	1650	1185	1060	620
11	14		1380	-1210	455	-1075	2765	2060	1385	1300	675
12	15		1620	-1340	580	-1205	2790	2300	1585	1450	725
13	16		1925	-1380	740	-1245	2550	2740	1920	1710	785
14	17		2035	-1680	800	-1350	2480	2880	2180	1880	835
15	18		2095	-2250	800	-1420	2460	2990	2460	2180	875
16	19		2150	-4700	790	-1700	3315	3060	2900	2470	955
17	20		2075	-8400	800						980

TABLE B.26.2 CENTERLINE STRAIN DISTRIBUTION AND BEAM DEFLECTIONS

Inc.	Strain (in/in x 10^5)							
	1	2	3	5	6	7	8	9
i	0	0	0	0	0	0	0	0
ii	88	76	68	19	- 6	- 8	- 8	- 7
1	160	135	125	42	11	10	9	6
2	154	131	122	47	15	12	13	12
3	142	123	117	50	22	20	19	18
4	128	111	110	54	30	28	28	27
5	118	103	104	56	38	36	35	36
6	110	97	97	56	44	41	43	41
7	106	92	93	55	49	47	45	46
8	99	82	81	54	57	54	53	52
9	- 8	10	27	50	67	63	62	62
10	-195	-80	-14	40	78	74	75	73
11	-453	-175	-86	30	93	88	88	90
12	-737	-254	-94	18	107	103	101	101
13	-886	-334	-165	0	128	123	123	123
14	-1006	-416	-240	- 1	146	142	139	141
15	-1314	-556	-419	-55	168	165	162	163
16		-1205	-1212	-171	232	232	226	229
17			-1458	-410	336	336	329	322

(i) Before Release; (ii) After Release; (-) Tension

Inc.	Load (kips)	Deflection (in)		
		North	℄	South
1	0.15	0.00	0.00	0.00
2	2	.04	.04	.04
3	4	.09	.09	.09
4	6	.15	.15	.15
5	8	.22	.22	.22
6	9	.27	.28	.27
7	10	.32	.33	.32
8	11	.41	.41	.41
9	12	.50	.51	.50
10	13	.61	.62	.61
11	14	.77	.79	.77
12	15	.90	.94	.89
13	16	1.12	1.15	1.12
14	17	1.32	1.34	1.31
15	18	1.58	1.59	1.56
16	19	2.20	2.22	2.18
17	20	3.11	3.06	3.03

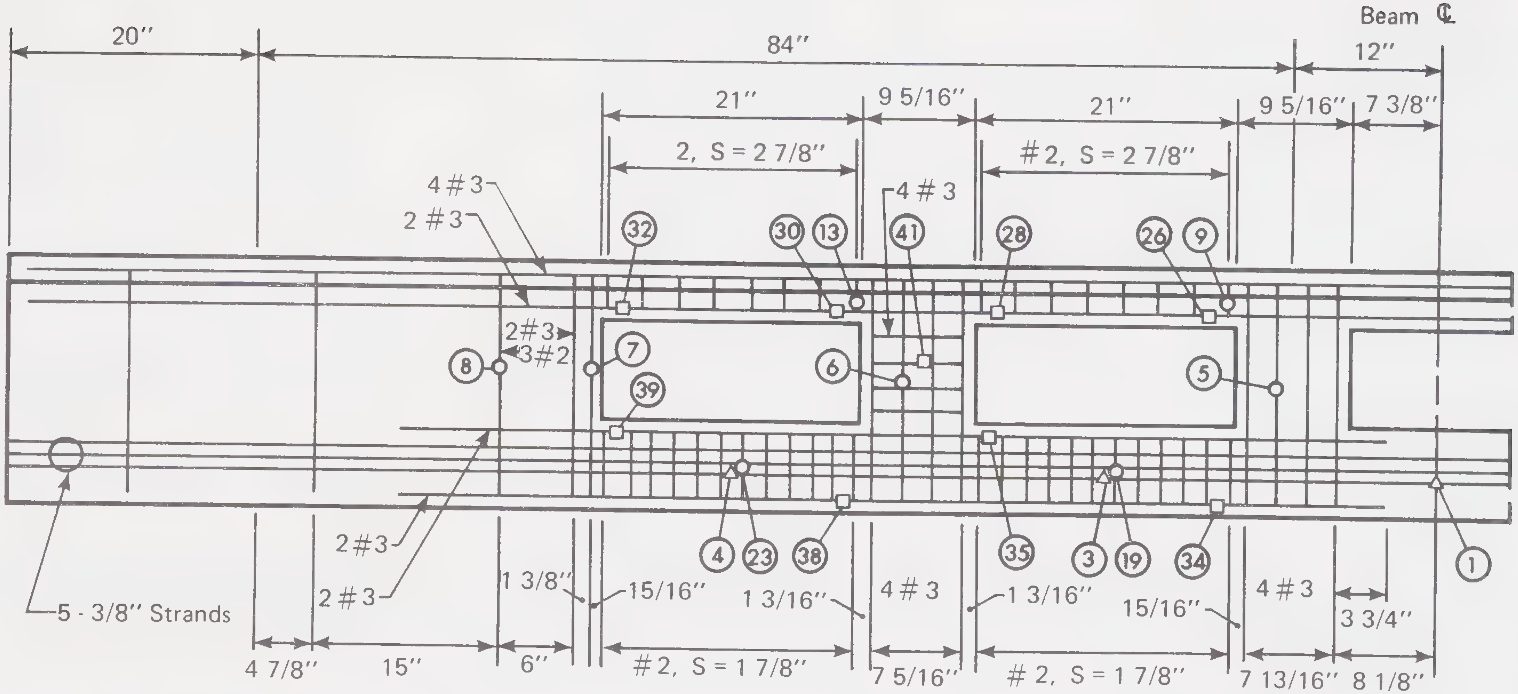


FIGURE B.26. Reinforcement Details and Strain Gage Locations for Beam 26–21–7

TABLE B.27.1 ELECTRICAL STRAIN GAGE MEASUREMENTS (MICRO INCHES PER INCH)

BEAM NO. 27-16-4

Inc. No.	Load per Jack	Strain Gage Number							
		1	4	6	7	8	8A	13	14
1	0.169	0	0	0	0	0	0	0	0
2	2	60	35	-15	15	10	0	5	30
3	4	105	60	-25	20	20	- 5	5	70
4	6	150	85	-35	0	30	- 5	0	90
5	8	200	120	-40	25	25	- 5	5	100
6	10	250	160	-45	120	15	0	15	115
7	12	310	220	-30	190	20	0	35	150
8	14	37[illegible]	450	0	500	815	35	120	235
9	16	580	575	-15	610	1000	30	175	270
10	18	1045	630	-10	680	1115	35	350	405
11	20	1525	695	630	770	1240	40	455	530
12	22	2085	790	695	865	1340	40	550	660
13	24	2565	870	740	950	1435	40	630	775
14	26	3215	960	1020	1080	1565	45	680	850
15	28	3972	1025	1085	1170	1665	50	715	875
16	30	4730	1130	1270	1280	1785	45	745	880
17	32	5910	1225	1930	1312	1910	40	775	895
18	34	7600	1325		1380	2005	30	810	920
19	36	9150	1400		1400	2075	30	820	930
20	38		1450		1395	2095	35	820	930

Inc. No.	Load per Jack	Strain Gage Number							
		23	30	32	38	39			
1	0.169	0	0	0	0	0			
2	2	- 5	30	-50	75	40			
3	4	- 5	60	-85	145	105			
4	6	- 5	100	-135	255	220			
5	8	5	150	-180	385	335			
6	10	35	225	-235	545	440			
7	12	95	330	-300	690	625			
8	14	325	505	-360	880	710			
9	16	555	580	-420	1040	830			
10	18	670	670	-480	1220	965			
11	20	740	745	-565	1340	1150			
12	22	800	880	-610	1485	1285			
13	24	860	1010	-650	1665	1450			
14	26	915	1180	-665	1785	1635			
15	28	975	1320	-635	1880	1820			
16	30	1020	1455	-600	2015	1985			
17	32	1075	1595	-560	2125	2145			
18	34	1115	1740	-510	2225	2215			
19	36	1135	1845	-415	2285	2370			
20	38	1130	1920	-315	2300	2430			

TABLE B.27.2 CENTERLINE STRAIN DISTRIBUTION AND BEAM DEFLECTIONS

Inc.	Strain (in/in x 10^5)							
	1	2	3	5	6	7	8	9
i	0	0	0	0	0	0	0	0
ii	97	74	54	8	- 4	- 1	0	- 3
1	244	215	155	66	35	36	37	35
2	236	212	150	68	38	41	43	41
3	231	208	147	70	43	45	47	44
4	224	103	146	72	47	50	53	49
5	218	198	142	74	50	54	57	53
6	210	192	139	75	53	58	61	58
7	204	187	134	76	58	63	66	63
8	191	180	130	77	63	68	70	68
9								
10	95	125	103	77	73	78	83	80
11	28	73	63	69	89	94	98	96
12	-24	48	-24	58	100	106	110	108
13	-51	15	-63	53	110	115	118	118
14	-95	-27	-92	45	121	127	132	131
15	-140	-69	-124	37	133	138	143	142
16	-204	-146	-181	25	147	153	159	155
17	-279	-237	-243	8	163	171	175	173
18	-396	-381	-359	-19	187	195	201	198
19	-677	-635	-610	-73	229	237	243	239
20	-1004	-1396	-1476	-226	290	302	309	303
(i) Before Release; (ii) After Release; (-) Tension								

Inc.	Load (kips)	Deflection (in)		
		North	℄	South
1	0.17	0.00	0.00	0.00
2	2	.04	.04	.03
3	4	.06	.07	.05
4	6	.09	.10	.08
5	8	.11	.31	.11
6	10	.14	.17	.13
7	12	.18	.21	.17
8	14	.22	.26	.21
9	16	.27	.31	.26
10	18	.33	.39	.32
11	20	.44	.57	.45
12	22	.56	.75	.57
13	24		.93	
14	26	.85	1.14	.87
15	28	.99	1.34	1.03
16	30	1.18	1.62	1.23
17	32	1.41	1.95	1.48
18	34	1.71	2.41	1.82
19	36	2.34	3.41	2.56
20	38	3.80	6.09	4.53

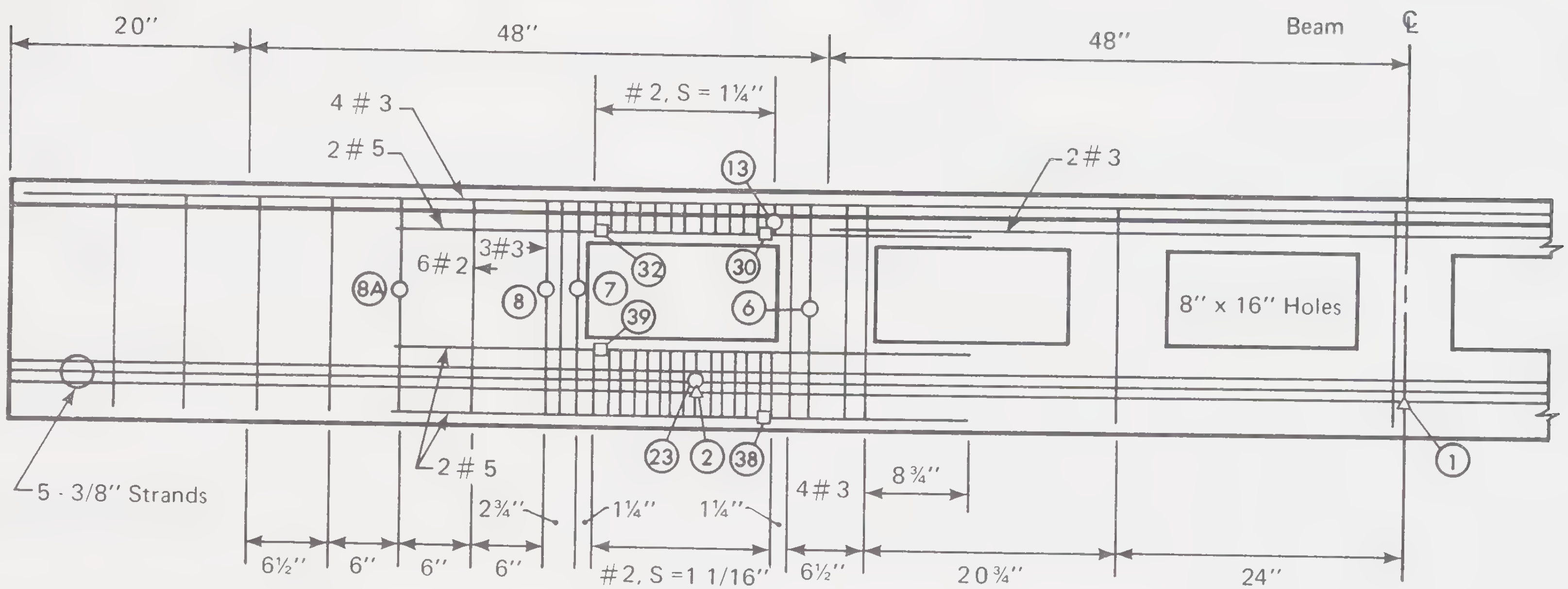


FIGURE B.27. Reinforcement Details and Strain Gage Locations for Beam 27–16–4

TABLE B.28.1 ELECTRICAL STRAIN GAGE MEASUREMENTS (MICRO INCHES PER INCH)

BEAM NO. 28-16-4

Inc. No.	Load per Jack	Strain Gage Numbers								
		1	4	6	7	8	8A	13	14	23
1	0.17	0	0	0	0	0	0	0	0	0
2	2.0	60	40	0	15	15	0	25		0
3	4.0	100	65	-10	30	25	0	105		- 5
4	6	140	95	-10	20	35	0	140		-10
5	8	180	130	-15	15	35	0	140		-10
6	10	230	170	-15	70	40	5	150		0
7	12	280	225	-20	140	60	5	160		20
8	14	340	610	-15	215	90	10	215		550
9	16	460	780	550	540	440	45	320		690
10	18	940	890	685	645	585	50	470		815
11	20	1460	1010	790	785	730	50	660		900
12	22	2095	1080	825	925	835	55	875		1015
13	24	2720	1085	875	1110	930	55	925		1110
14	26	3365	1300	890	1275	1065	50	960		1220
15	28	4155	1440	915	1475	1275	45	990		1355
16	30		1590	955	1600	1460	100	1020		1460
17	32		1715	1000	1720	1685	820	1055		1540
18	34		1850	975	1810	1750	1055	1090		1520
19	36		1965	875	1870	1800	1405	1100		1575
20	38		2005	785	1880	1830	1860	1110		1660

Inc. No.	Load per Jack	Strain Gage Numbers								
		30	32	38	39					
1	0.17	0	0	0	0					
2	2.0	5	-45	70	40					
3	4.0	25	-85	130	85					
4	6	50	-130	230	195					
5	8	70	-180	340	295					
6	10	105	-240	475	400					
7	12	150	-300	645	535					
8	14	270	-375	820	720					
9	16	400	-430	1000	825					
10	18	430	-540	1190	1025					
11	20	445	-610	1355	1185					
12	22	450	-660	1570	1365					
13	24	530	-705	1685	1535					
14	26	606	-720	1800	1730					
15	28	675	-720	1930	1910					
16	30	710	-680	2050	2110					
17	32	750	-580	2195	2300					
18	34	800	-440	2300	2470					
19	36	850	-290	2380	2635					
20	38	865	-160	2415	2720					

TABLE B.28.2 CENTERLINE STRAIN DISTRIBUTION AND BEAM DEFLECTIONS

Inc.	Strain (in/in x 10^5)							
	1	2	3	5	6	7	8	9
i	0	0	0	0	0	0	0	0
ii	85	68	49	13	- 2	0	- 1	- 3
1	221	189	147	75	34	44	40	34
2	215	186	143	77	40	50	45	40
3	211	183	143	79	44	55	49	45
4	205	177	140	81	47	58	53	49
5	198	173	137	82	52	63	57	53
6	191	167	133	84	57	67	63	52
7	183	162	131	86	60	73	67	62
8	175	157	128	88	65	77	72	67
9	156	145	121	88	72	84	78	73
10	85	106	107	87	81	94	87	83
11	23	56	83	81	94	116	98	96
12	-54	2	52	66	110	122	115	112
13	-133	-55	21	61	120	132	123	122
14	-211	-120	-18	56	133	144	135	135
15	-283	-183	-60	53	146	158	148	150
16	-444	-297	-124	45	163	175	166	165
17	-654	-450	-212	28	184	197	186	187
18	-1144	-860	-440	-35	234	249	223	243
19		-1730	-895	-166	317	332	316	329
20			-1310	-212	348	360	342	358

(i) Before Release; (ii) After Release; (-) Tension

Inc.	Load (kips)	Deflection (in)		
		North	℄	South
1	0.17	0.00	0.00	0.00
2	2	.03	.03	.03
3	4	.05	.06	.05
4	6	.07	.09	.08
5	8	.10	.13	.11
6	10	.13	.17	.13
7	12	.16	.21	.18
8	14	.22	.26	.22
9	16	.27	.31	.27
10	18	.35	.45	.36
11	20	.46	.62	.50
12	22	.61	.83	.64
13	24	.76	1.02	.77
14	26	.93	1.23	.94
15	28	1.11	1.50	1.11
16	30	1.33	1.81	1.35
17	32	1.64	2.25	1.68
18	34	2.50	3.53	2.62
19	36	4.03	5.87	4.22
20	38	5.01	7.56	5.16

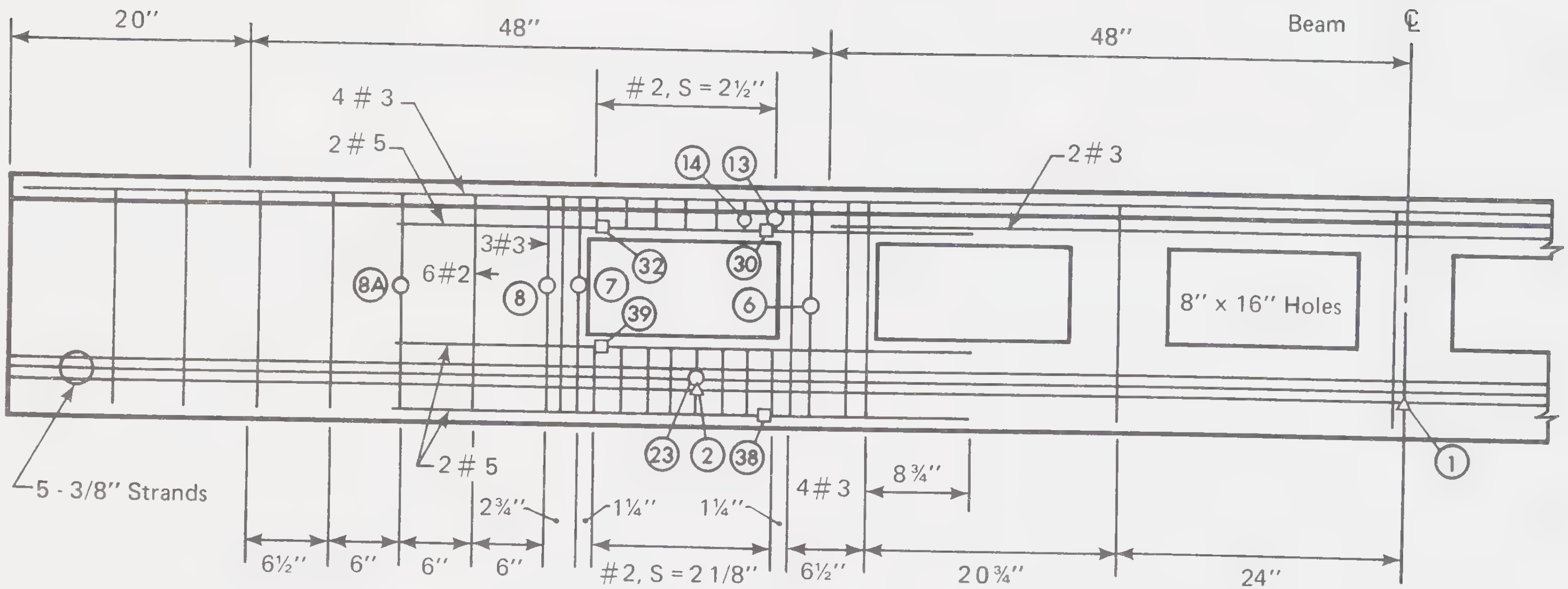


FIGURE B.28. Reinforcement Details and Strain Gage Locations for Beam 28–16–4

TABLE B.29.1 ELECTRICAL STRAIN GAGE MEASUREMENTS (MICRO INCHES PER INCH)

BEAM NO. 29-12-6

Inc. No.	Load per Jack	Strain Gage Numbers								
		1	3	4	5	6	6A	7	9	13
1	0.15	0	0	0	0	0	0	0	0	0
2	2.0	60	40	20	- 5	0	-20	5	10	10
3	4	140	95	50	-10	5	-65	10	20	25
4	6	220	155	85	-20	20	-115	10	35	30
5	8	300	210	110	-25	50	-165	5	45	35
6	9	355	255	135	-20	80	-195	10	45	35
7	10	410	300	150	-15	105	-215	20	45	30
8	11	520	365	175	-10	125	-225	35	50	50
9	12	700	440	190	0	1245	740	220	70	80
10	13	840	520	210	5	1340	835	250	90	100
11	14	1310	770	245	10	1590	1010	310	155	120
12	15	1830	1150	270	20	1910	1140	350	230	135
13	16	2440	1550	330	25	4050	1380	545	295	150
14	17	2905	1835	360	0	6000	1530	620	325	175
15	18	3490	2250	750	- 5		1690	675	445	220
16	19	4355	2800	975	-15		1830	755	610	260
17	20									

Inc. No.	Load per Jack	Strain Gage Numbers									
		19	23	26	28	30	32	34	35	38	39
1	0.15	0	0	0	0	0	0	0	0	0	0
2	2.0	- 5	- 5	- 5	-45	0	-35	75	40	45	35
3	4	-15	-10	-10	-110	10	-90	185	160	115	85
4	6	-25	-20	-15	-185	20	-150	310	175	170	150
5	8	-35	-25	-10	-260	25	-210	440	250	260	205
6	9	-50	-30	- 5	-330	20	-260	565	330	315	250
7	10	-65	-35	0	-410	15	-300	700	380	350	290
8	11	-80	-40	25	-550	15	-350	920	505	415	350
9	12	-85	-50	215	-625	-35	-445	1205	600	440	470
10	13	-85	-55	335	-725	-35	-530	1455	750	485	575
11	14	-70	-60	585	-920	-25	-610	1860	1080	580	745
12	15	-30	-70	830	-965	-15	-680	2080	1430	775	930
13	16	310	-90	1150	-975	30	-810	2245	1835	1245	1170
14	17	435	-80	1350	-990	55	-970	2310	2150	1635	1250
15	18	545	450	1590	-1000	110	-1190	2405	2720	1960	1465
16	19	635	580	1810	-1035	140	-1530	2570	3800	2440	1850
17	20		680							3150	2190

TABLE B.29.2 CENTERLINE STRAIN DISTRIBUTION AND BEAM DEFLECTIONS

Inc.	Strain (in/in x 10^5)							
	1	2	3	4	5	6	7	8
i	0	0	0	0	0	0	0	0
ii	84	65	43	9	15	- 2	1	- 1
1	187	154	108	38	45	32	35	36
2	183	151	107	34	46	38	42	41
3	172	143	102	33	48	44	48	48
4	162	135	98	32	51	51	55	55
5	150	128	93	34	53	58	61	61
6	143	122	88	33	55	61	65	65
7	136	117	86	34	56	64	68	68
8	120	108	80	33	58	70	74	74
9	92	93	72	34	58	73	77	76
10	76	80	64	34	59	79	83	83
11	23	40	34	31	56	88	91	91
12	-37	- 4	-11	26	54	97	99	98
13	-94	-60	-67	- 4	48	107	108	108
14	-137	-107	-109	-15	43	117	118	117
15	-185	-164	-158	-32	37	127	128	128
16	-266	-248	-231	-59	30	139	140	140
17	-360	-340	-309	-87	19	151	151	

(i) Before Release; (ii) After Release; (-) Tension

Inc.	Load (kips)	Deflection (in)		
		North	℄	South
1	0.15	0.00	0.00	0.00
2	2	.03	.03	.03
3	4	.07	.08	.07
4	6	.12	.12	.11
5	8	.16	.16	.15
6	9	.19	.19	.18
7	10	.22	.22	.21
8	11	.25	.26	.24
9	12	.30	.31	.28
10	13	.35	.36	.33
11	14	.46	.49	.46
12	15	.56	.60	.55
13	16	.70	.75	.69
14	17	.83	.88	.81
15	18	.97	1.04	.96
16	19	1.19	1.28	1.18
17	20	1.40	1.69	1.39

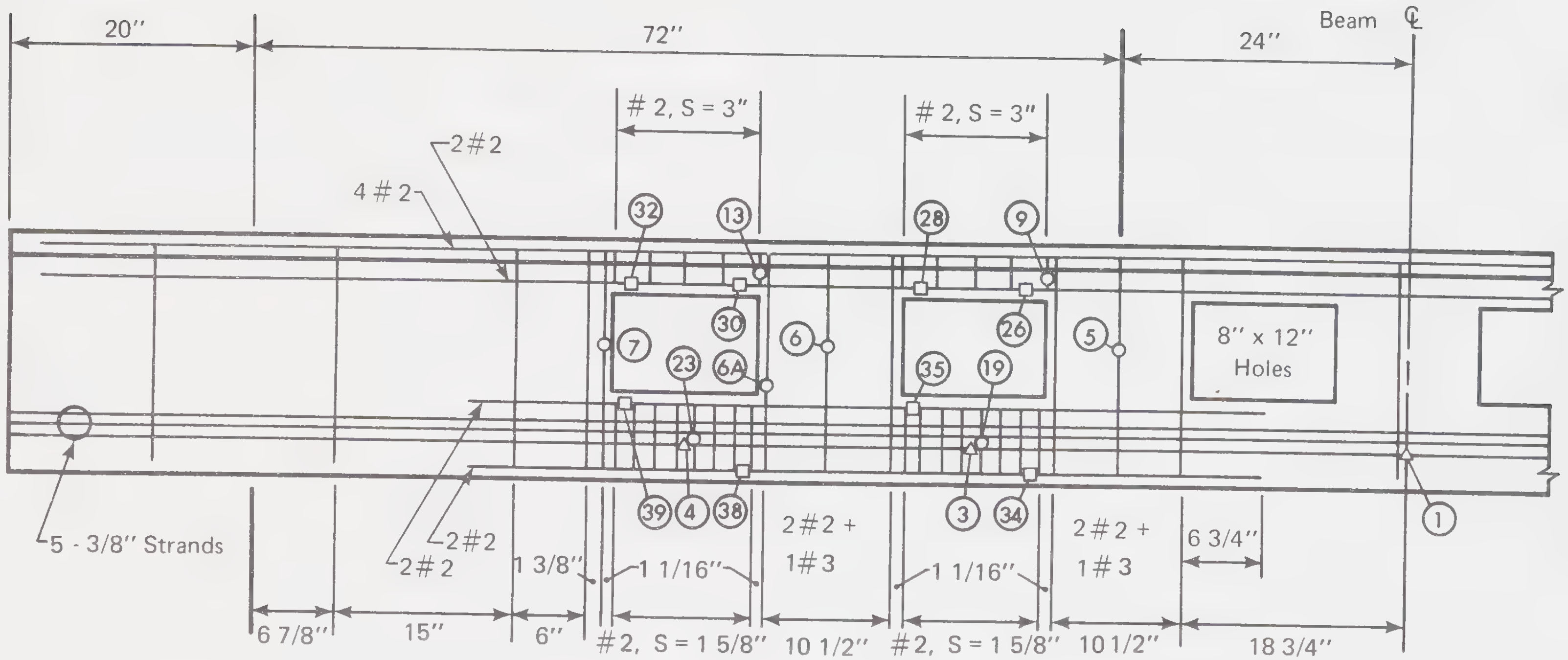


FIGURE B.29. Reinforcement Details and Strain Gage Locations for Beam 29–12–6

TABLE B.30.1 ELECTRICAL STRAIN GAGE MEASUREMENTS (MICRO INCHES PER INCH)

BEAM NO. 30-21-7

Inc. No.	Load per Jack	Strain Gage Numbers								
		1	3	4	5	6	7	9	13	19
1	0.15	0	0	0	0	0	0		0	0
2	2	65	45	20	0	20	10		10	-10
3	4	160	110	50	0	60	30		65	-20
4	6	260	190	80	10	745	25		135	-35
5	8	355	300	110	20	1025	330		160	-10
6	9	425	550	130	35	1160	420		180	160
7	10	510	780	145	35	1275	495		195	335
8	11	1575	920	170	60	1415	615		240	455
9	12	2050	1040	190	65	1530	675		270	570
10	13	2580	1180	210	685	1570	740		310	700
11	14	3205	1355	235	815	1730	830		415	1050
12	15	3810	1535	325	850	1885	895		425	955
13	16	1170	1705	740	940	2015	1100		650	1400
14	17	1900	1820	830	970	2180	1220		675	
15	18	1750	1935	930	970	2350	1335		695	
16	19					2500				

Inc. No.	Load per Jack	Strain Gage Number									
		23	26	28	30	32	34	35	38	39	41
1	0.15	0	0	0	0	0	0	0	0	0	0
2	2	-10	0	-55	10	-45	85	45	60	45	0
3	4	-15	35	-140	35	-115	230	130	140	135	10
4	6	-20	135	-230	50	-220	495	300	230	410	540
5	8	-25	270	-335	115	-320	850	490	360	595	750
6	9	-35	390	-410	145	-385	1075	675	520	720	865
7	10	-40	500	-475	175	-450	1250	825	650	820	960
8	11	-45	730	-580	225	-550	1465	1045	850	995	1080
9	12	-50	865	-640	270	-605	1650	1190	980	1130	1155
10	13	5	930	-720	325	-685	1830	1390	1130	1270	1215
11	14	30	1150	-805	410	-790	2005	1640	1340	1435	1345
12	15	100	1235	-860	430	-870	2180	1805	1520	1580	1440
13	16	625	1400	-905	250	-1010	2440	1980	1770	1855	1575
14	17	700	1500	-950	210	-1100	2640	2030	1930	2030	1690
15	18	780	1635	-965	170	-1190	2810	2200	2075	2200	1850
16	19										

TABLE B.30.2 CENTERLINE STRAIN DISTRIBUTION AND BEAM DEFLECTIONS

Inc.	Strain (in/in x 10^5)							
	1	2	3	5	6	7	8	9
i	0	0	0	0	0	0	0	0
ii	91	77	68	21	- 3	- 1	- 3	- 2
1	216	170	156	63	24	26	29	27
2	210	166	155	65	29	31	36	34
3	197	155	158	67	38	40	43	42
4	182	145	142	70	47	50	52	54
5	172	135	134	69	56	59	61	66
6	161	125	125	69	64	66	67	73
7	154	91	118	69	69	73	74	81
8	102	44	112	63	80	84	84	92
9	-12	- 3	99	60	90	93	93	103
10	-270	-75	75	52	101	104	103	115
11	-401	-157	-72	40	116	120	129	134
12	-627	-169	-246	19	130	136	133	147
13	-897	-239	-465	- 6	150	155	154	170
14	-1190	-319	-649	-27	167	174	171	189
15	-1530	-457	-906	-49	187	193	191	209
16								

(i) Before Release; (ii) After Release; (-) Tension

Inc.	Load (kips)	Deflection (in)		
		North	℄	South
1	0.15	0.00	0.00	0.00
2	2	.04	.04	.04
3	4	.09	.09	.08
4	6	.17	.17	.16
5	8	.25	.24	.24
6	9	.31	.30	.30
7	10	.37	.36	.36
8	11	.47	.48	.46
9	12	.56	.57	.55
10	13	.66	.69	.64
11	14	.80	.84	.80
12	15	.95	.97	.93
13	16	1.14	1.20	1.12
14	17	1.30	1.33	1.29
15	18	1.48	1.49	1.45
16	19		1.89	

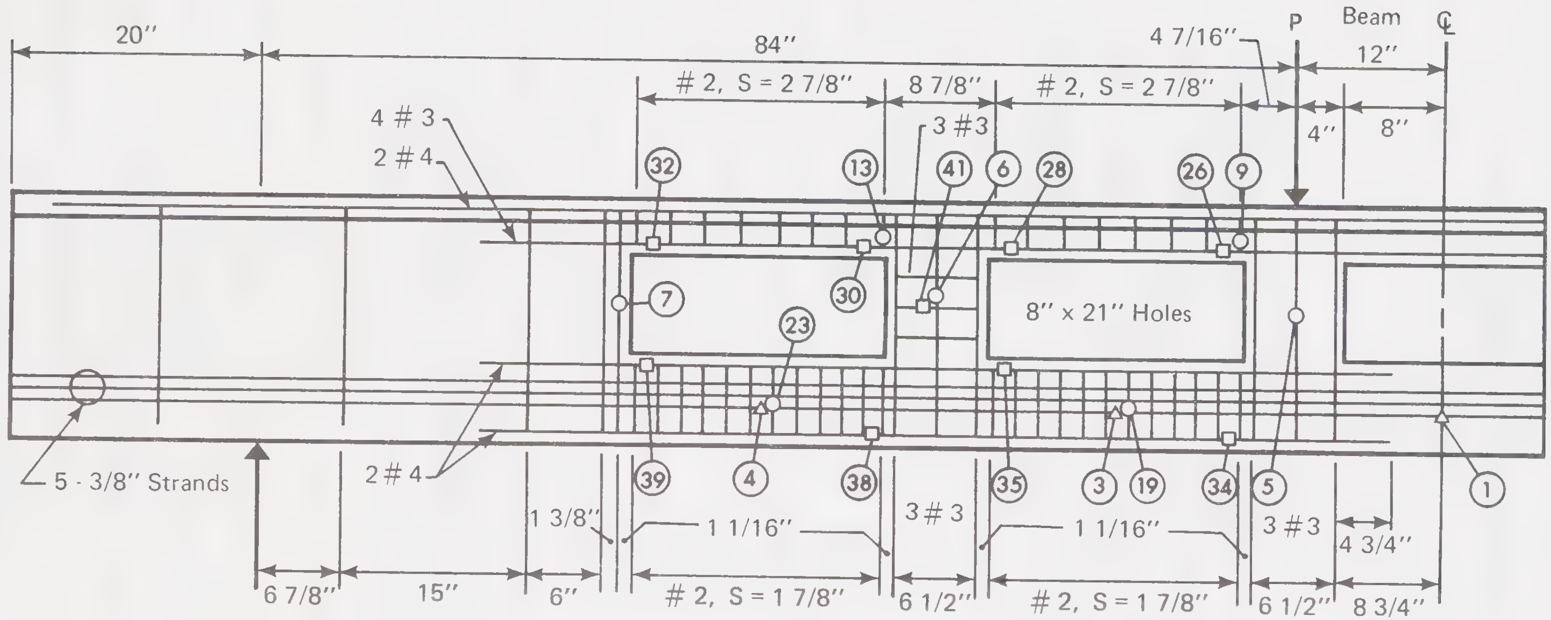


FIGURE B.30. Reinforcement Details and Strain Gage Locations for Beam 30–21–7

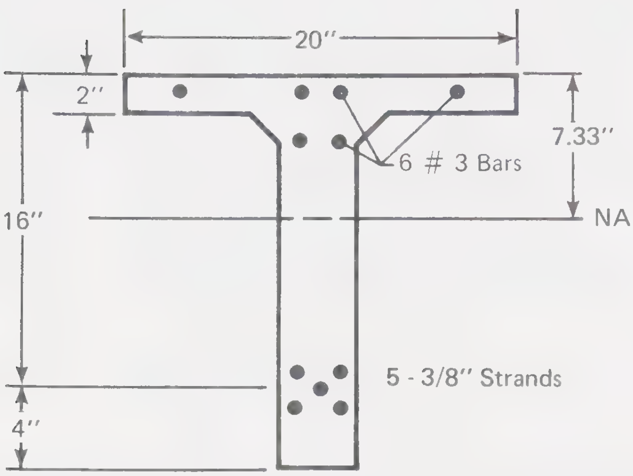
APPENDIX - C

DESIGN EXAMPLE AND NOTATION

C.1 Design

In the design example below, beam 26-21-7 is designed in detail following the design procedure outlined in Chapter 6. In the design of the other test beams, the same procedure was followed to varying degrees. The flexural design of the main section as outlined in the example was used for all of the test beams and is based on the ACI 318-71 Code (1). The design of the reinforcement in the shear spans deviated in some cases from the procedure outlined in the example. For beams 1-16-6 to 16-12-6, the selection of the shear reinforcement for the posts, struts and solid shear spans was based on; (i) calculations from the ACI 318-71 Code (1) with $b_w d = 32$ or $16\ in^2$ for the solid sections and the sections through the holes, respectively, (ii) the experience of the author, and (iii) the results of the tests of LeBlanc and Sauve. The supplementary longitudinal reinforcement for these first 16 beams was selected on the basis of experience. For beams 17-16-4 to 30-21-7, the design procedure used for each beam was similar to that in the example. There were, however, modifications in the design procedure and the selection of reinforcement. Modifications to the design consisted of changing the proportion of the shear carried by the struts above and below an opening. The reinforcement at various locations in the shear spans was altered to examine the effect of these changes on the beam behaviour.

C.2 Design Example



Beam 26-21-7

$A_g = 114.25 \text{ in}^2$
$I_g = 4583 \text{ in}^4$

$f_c' = 5274$ psi
$f_{sp} = 410$ psi

f_y #2 Stirrups = 33.8 ksi
f_y #3 Stirrups = 52.2 ksi
f_y #3 Long. = 51.4 ksi

$f_{su} = 275.625$ ksi
$f_{pi} = 178.08$ ksi
$f_{se} = 137.04$ ksi
$E_{ps} = 28.9 \times 10^3$ ksi
$A_{ps} = 5 \times 0.08$
$= 0.40 \text{ in}^2$

1. Stresses in the concrete at transfer.

    At top over support .... 364 psi tension
    At top at mid-span .... 291 psi tension
    At bottom at mid-span .... 2204 psi compression

2. Working load moment (based on flexural cracking).

$$M_{cr} = \frac{I_g}{y} (f_{sp} + f_{re} - f_d)$$

$$= \frac{4583}{12.67} (410 + 1794 + 126)$$

$$= 752 \text{ in-k}$$

3. Ultimate flexural capacity (based on the ACI 318-71 Code equation for the stress in the prestressing strand and the Whittney stress block).

$$f_{ps} = f_{pu} \left(1 - 0.5\, \rho_p \frac{f_{pu}}{f_c'}\right) \qquad \text{(ACI Equation 18-3)}$$

$$\rho_p = \frac{A_{ps}}{bd} = 0.00125$$

thus f_{ps} = 266.60 ksi. Assuming the two No. 3 bars running the length of the beam at a depth of 3.50 inches yield in tension (this was checked and found to be true):

$$M_u = \phi[f_{ps} A_{ps} (d_1 - \frac{a}{2}) + f_y A_s (d_2 - \frac{a}{2})]$$

for $a = T/0.85\, f_c'b$ = 1.31 inches and $\phi = 1$:

$$M_u = 1669 \text{ in-k}$$

The maximum load per jack to reach the flexural capacity is then:

$$P_u = \frac{M_u}{84} = 19.9 \text{ k}$$

4. The shear reinforcement for the solid shear spans was designed to resist the applied shear associated with flexural failure using Chapter 11 of the ACI 318-71 Code with $\phi = 1$. Extra stirrups were placed beside the first holes in each shear span.

The shear strength of the concrete section as calculated by Section 11.5 of the ACI 318-71 Code was greater than the applied shear stresses so the maximum spacing (15 inches) and the minimum allowable area of shear reinforcement (0.08 in^2) governed. Double-legged No. 2 stirrups were used spaced at 15 inches.

Extra shear reinforcement designed to carry the total shear force was

placed beside the first hole in each shear span. This required a stirrup area of 0.36 in^2 and the two double-legged No. 3 stirrups used provided an area of 0.44 in^2.

5. Design of the struts in the shear spans. Assuming:

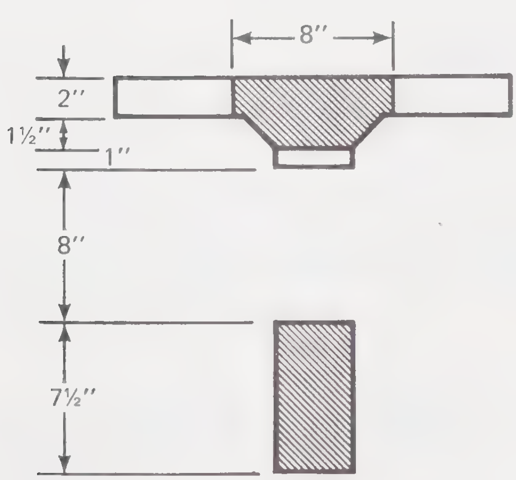
   (i) Vierendeel truss action of the beam in the region of an opening.

   (ii) Points of inflection are located at the center of the strut above and below an opening.

   (iii) The shear at the ultimate load is proportioned to the struts according to their respective shear areas (which includes the contribution of the flange to the shear area of the top strut).

   (iv) The axial force in the struts is equal to the bending moment at the centerline of an opening divided by the distance between the centroid of the top strut and the centroid of the primary tension reinforcement.

There are two openings in each shear span but the axial loads were higher above and below hole 2 so the reinforcement was designed for this hole and the same reinforcement was placed in the struts of both openings. For a section through a hole, the properties are given below.



Top Strut

A_g = 52.25 in^2
I_g = 62.7 in^4
y_{top} = 1.49 in
Shear area crosshatched = 24.5 in^2

Bottom Strut

A_g = 30.00 in^2
I_g = 140.6 in^4
y_{top} = 3.75 in^2
Shear area crosshatched = 30.0 in^2

Shear in the top strut, $V_{Top} = \frac{24.5}{24.5 + 30.0}$, $p_u = 8.96$ k.

Shear in the bottom strut, $V_{Btm} = \frac{30}{24.5 + 30.0} \cdot p_u = 10.94$ k.

Axial force in the top strut is = 93.79 k.

The effective axial force in the bottom strut is the axial force in the top strut less the effective prestress force

$$= 93.79 - A_{ps} f_{pe}$$

$$= 38.97 \text{ k}$$

The maximum moments at the ends of the struts are:

$$M_{u\ Top} = 8.96 \times \ell n/2 = 94.1 \text{ in-k}$$

$$M_{u\ Btm} = 10.94 \times \ell n/2 = 114.9 \text{ in-k}$$

5a. Strut shear reinforcement design. For the top strut, strength calculations based on equation 11-7 of the ACI 318-71 Code including the effect of the axial load gave a maximum spacing of 9.28 inches for double-legged No. 2 stirrups. The maximum allowable spacing of 0.75 h was 3.4 inches. Therefore, double-legged No. 2 stirrups were used spaced at 3.0 inches.

For the bottom strut, the strength calculations of equation 11-8 of the ACI 318-71 Code, which also includes the effect of axial load, gave a spacing of 2.01 inches for double-legged No. 2 stirrups. Therefore, double-legged No. 2 stirrups were used spaced at 1.8 inches.

5b. Design struts for axial load and moment.

<u>Top Strut</u>

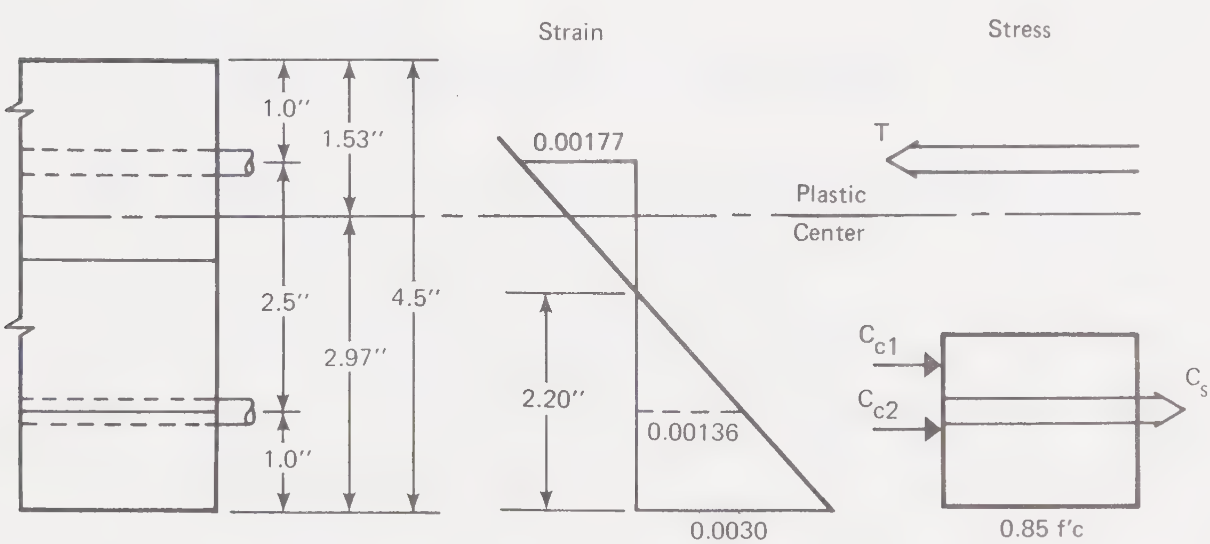
Try six No. 3 bars as longitudinal reinforcement.

Draw the interaction diagram for axial load and moment.
Locate the plastic centroid.

Taking moments about C_c, the plastic centroid was found to be 1.53 inches below the top of the section and the maximum axial load the section could carry was found to be 268 kips.

For balanced failure (compression on the bottom):

$$\varepsilon_c = 0.0030$$
$$\beta_1 = 0.79$$
$$\varepsilon_y = 0.00177$$
$$f_y = 51.4 \text{ ksi}$$
$$E_s = 29.1 \times 10^3 \text{ ksi}$$



$$c = \frac{3.5}{0.00177 + 0.003} \times 0.003 = 2.20 \text{ in}$$

Then:

$$\varepsilon_s' = \frac{0.003}{2.20} \times 1.0 = 0.00136$$

Therefore:

$$C_s = 8.71 \text{ k}$$

Acting 1.97 inches below the plastic center.

The compressive force in the concrete was divided into two parts; C_{c1} acting on the rectangular portion, and C_{c2} acting on the chamfers.

$$a = \beta_1 c = 1.74 \text{ in}$$

then:

$$C_{c1} = 0.85 \times f_c' \times b \times a = 31.20 \text{ k}$$

Acting 2.10 inches below the plastic center.

and:

$$C_{c2} = 2.45 \text{ k}$$

Acting 1.48 inches below the plastic center.

The total tensile force provided by the four No. 3 bars is T.

$$T = 51.4 \times 4 \times .11 = 22.62 \text{ k}$$

Acting 0.59 inches above the plastic center.

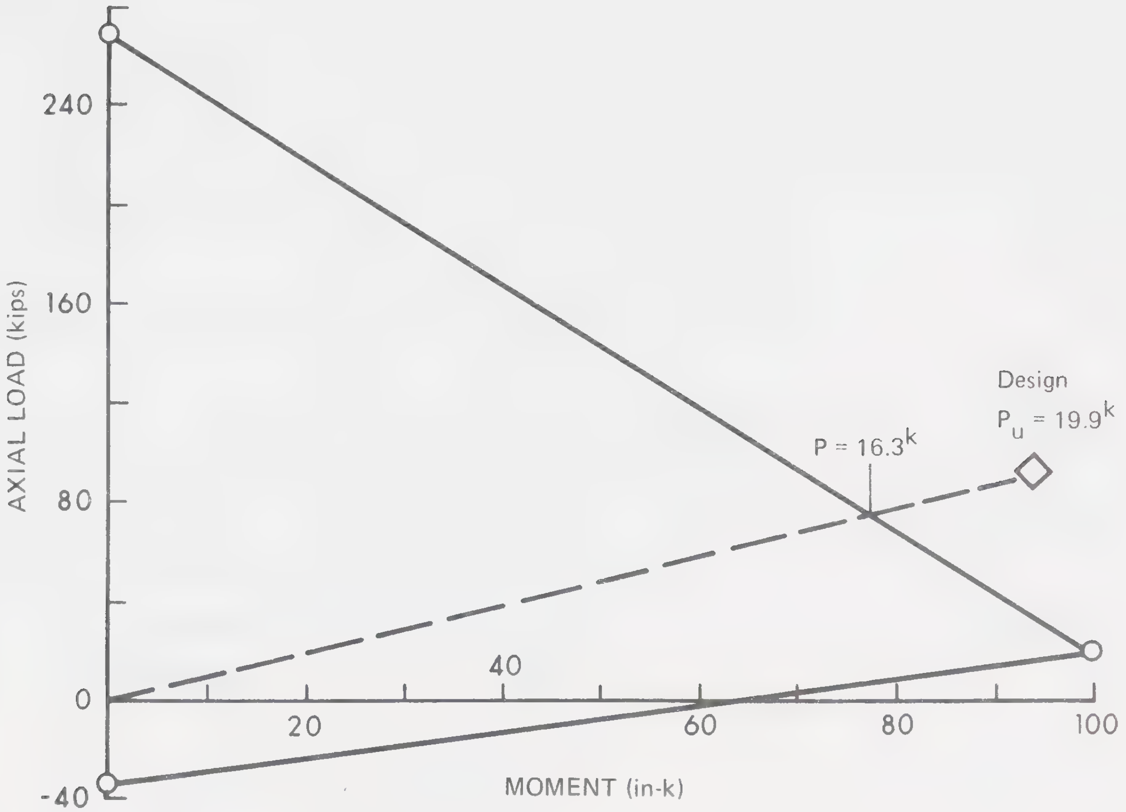
Totaling the axial force and moments about the plastic center gives

$$P_b = 19.74 \text{ k}$$

$$M_b = 99.65 \text{ in-k}$$

For the entire ϕ section in tension:

$$T = 6 \times 0.11 \times 51.4 = 33.92 \text{ k}$$



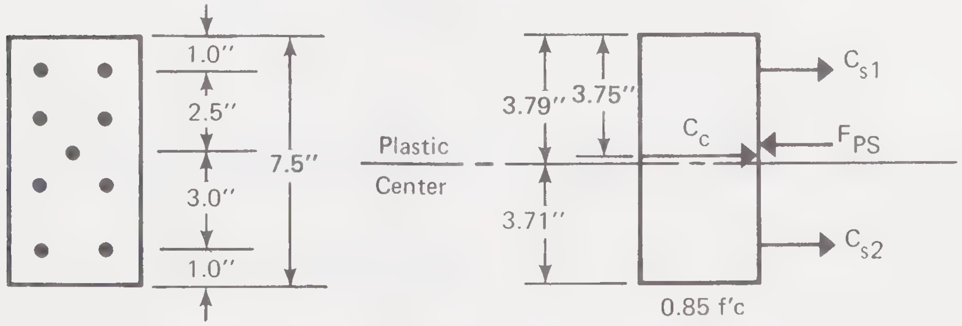
From the plotted interaction diagram, it was noted that the reinforcement was not sufficient to carry the full design load of 19.9 kips, however, six No. 3 bars were used.

Bottom Strut

Try four No. 3 bars as supplementary longitudinal reinforcement.

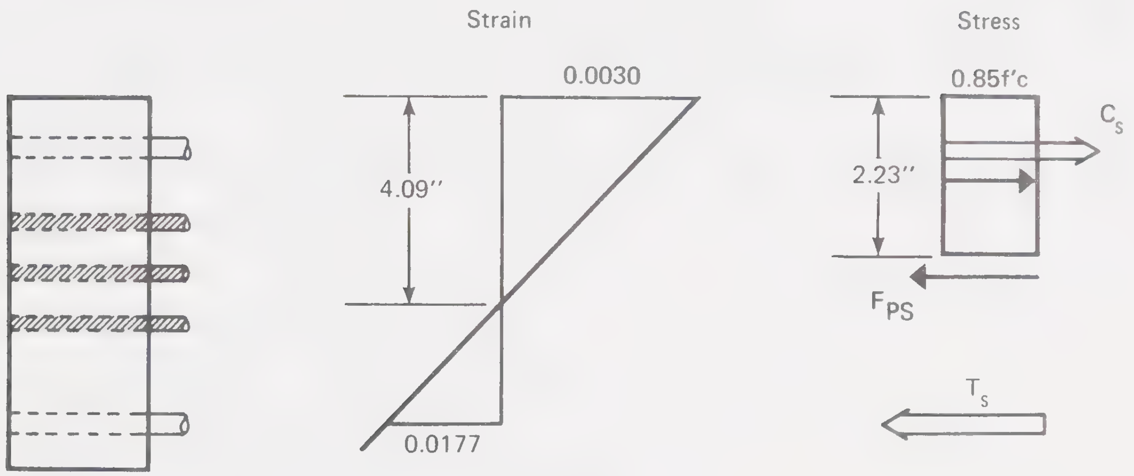
Draw the interaction diagram for the top strut.

Locate the plastic centroid.



Taking moments about C_c the plastic center was found to be 3.79 inches below the top of the section. The maximum compressive axial load was 137 kips.

for balanced failure (compression on the top)



$$c = \frac{0.003}{0.003 + 0.00177} \times 6.5 = 4.09 \text{ inches}$$

then:

$$\varepsilon'_{s1} = \frac{0.003}{4.09} \times 3.09 = 0.0023 > \varepsilon_y$$

therefore:

$$C_{s1} = 11.31$$

Acting 2.79 inches above the plastic center

$$\varepsilon_{ps} = \frac{0.0030}{4.09}(4.09 - 3.5) - \frac{137.04}{28.9 \times 10^3}$$

$$= 0.00431 \text{ tension}$$

$$F_{ps} = A_{ps} + \varepsilon_{ps} + E_{ps} = 49.82 \text{ k tension}$$

Acting 0.29 inches above the plastic center

$$a = \beta_1 c = 3.23 \text{ inches}$$

then:

$$C_c = 3.23 \times 4.0 \times 0.85 \times 5274 = 57.74 \text{ kips}$$

Acting 2.18 inches above the plastic center.

and:

$$T = 2 \times 0.11 \times 51.4 = 11.31 \text{ kips}$$

Acting 2.71 inches below the plastic center.

Totaling the axial force and moments about the plastic center gives:

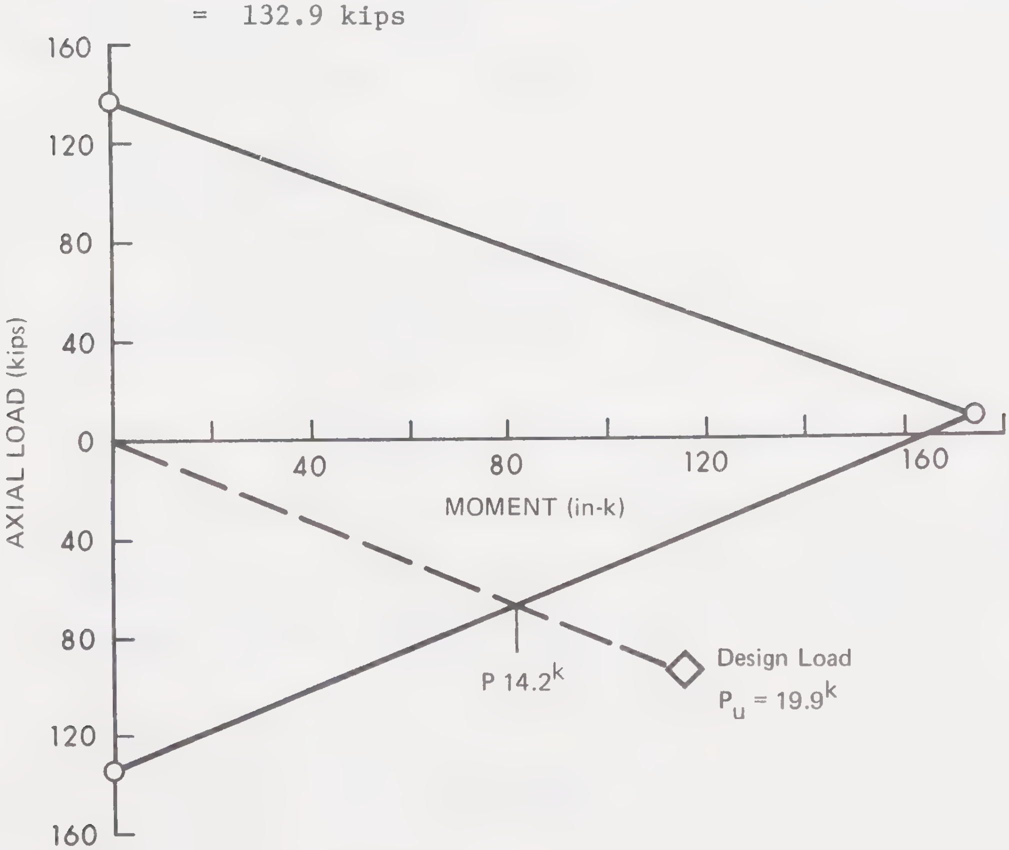
$$P_b = 8.12 \text{ kips}$$

$$M_b = 174 \text{ in-kips}$$

For the complete section in tension at rupture of the strands ignoring strain hardening in the mild steel;

$$T = 5 \times 0.08 \times 275.625 + 4 \times 0.11 \times 51.4$$

$$= 132.9 \text{ kips}$$



From the plotted interaction diagram, it was noted that the strut flexural reinforcement was inadequate but four No. 3 bars were used.

5c. Check the anchorage of the strut flexural reinforcement. From Section 12.5(a) of the ACI 318-71 Code, the basic development length in tension for a No. 3 bar with $d_b = 0.375$ in. $A_s = 0.11$ in^2 and $f_y = 51.4$ ksi:

$$0.04 \ A_b f_y / \sqrt{f_c'} = 3.1 \text{ inches}$$

but not less than:

$$0.0004 \ d_b f_y = 7.7 \text{ inches}$$

nor less than 12 inches.

For deformed bars in compression Section 12.6 of the ACI 318-71 Code the development length is:

$$0.2 \ f_y d_b / \sqrt{f_c'} = 5.3 \text{ inches}$$

but not less than:

$$0.0003 \ f_y d_b = 5.8 \text{ inches}$$

nor less than 12 inches.

Therefore, the minimum post and strut length is $8 + 12 = 20$ inches. The strut length (21 inches) is adequate but the 9 5/16 post length is less than required by the above calculation but from experience the post length is adequate.

6. Design the post reinforcement. The maximum shear on the section is $V_u = 19.9$ k. Calculate the horizontal shear on post 1 (V_h).

$$V_h = \frac{S_H V_H}{d_1}$$

$$= 41.57 \text{ k}$$

$d = 16 - 1.49 = 14.51$ in.

S_H = hole spacing
$= 21 + 9\ 5/16$
$= 30.31$ inches

then:

$$V_{uh} = \frac{41.57}{4 \times 9.31} = 1116 \text{ psi} = 15.4 \sqrt{f_c'}$$

Design vertical and horizontal stirrups to carry this total horizontal shear. The required area of reinforcement is:

$$A_s = \frac{V_h}{f_y} = 0.75 \text{ in}^2$$

Four double-legged stirrups were used both vertically and horizontally providing an area of 0.88 in^2.

C.3 Notation

a = depth of equivalent rectangular stress block

A_g = cross area of concrete section

A_{ps} = area of prestressed reinforcement

A_s = area of mild steel reinforcement

A_v = area of shear reinforcement

b = width of compression face

b_w = web width

c = distance from the extreme compression fiber to neutral axis

C = total compressive force

d = distance from the extreme compression fiber to the centroid of the tension reinforcement

d_1 = distance from the centroid of the compression strut to the centroid of the tension reinforcement

d_b = nominal diameter of a bar

E_c = modulus of elasticity of concrete

E_s = modulus of elasticity of steel

f_c' = compressive strength of concrete

f_d = stress due to dead loads

f_{pi} = stress in the prestressing strand before release

f_{pe} = compressive stresses in concrete due to prestress only after losses

f_{ps} = calculated stress in the prestressing strand at the ultimate load

f_{pu} = ultimate strength of prestressing steel

f_{se} = effective stress in prestressing steel after losses

f_{sp} = tensile splitting strength of concrete

f_y = yield strength of mild steel reinforcement

h = over all thickness of a member

I_g = moment of inertia of gross concrete section about the centroidal axis

ℓ_h = hole length

ℓ_d = development length

M_{cr} = cracking moment

M_u = ultimate flexural capacity

N_u = design axial load normal to the cross-section occurring simultaneously with V_u

p_u = ultimate applied load per jack

s = stirrup spacing

T	=	total tensile force
v_c	=	shear carried by the concrete
V_h	=	horizontal shear on a post at the ultimate load
V_u	=	ultimate shear force at a section
β_1	=	factor defined in section 10.2.7 of ACI 138-71
ε_c	=	strain in the concrete
ε_s	=	strain in the mild steel reinforcement
ε_y	=	yield strain for the mild steel reinforcement